WATER MANAGEMENT PLAN
FOR THE WESTERN C-9 BASIN

August, 1976

Prepared by:

Resource Planning Department
Central and Southern Florida Flood Control District

For:

South Florida Regional Planning Council

"This public document was promulgated at an annual cost of $386.00 or $.76 per copy to inform the public, governmental agencies and officials of the District's water management plan for the Western C-9 Basin." RPD-588 5C

Water Management Plan
For the Western C-9 Basin

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I. OBJECTIVES

Development of an integrated land and water management plan for the Canal 9 Basin must be predicated on the existing water resource limitations of that basin. The constraints of the existing system, in terms of storm water disposal, water supply, and water quality, form the basis for land development decisions to be made in the near future. The first stage in preparing an integrated land and water management plan must be the definition of these limitations and the implications of these constraints in terms of permissible land use intensity. This initial effort could thus provide the basis for development of short-term land use plans by the local governments involved, which would be consistent with the water resource constraints of the basin.

The South Florida Regional Planning Council, at its June, 1975 meeting, directed the Council staff to conduct a comprehensive study for a subregion of Broward and Dade counties. This subregion includes all of the western C-9 basin. As part of this overall effort, the District agreed to investigate the hydrological aspects of the western C-9 basin, specifically in terms of storm water disposal limitations, water supply capability, and, to a much lesser extent, water quality. The Council assumed the responsibility of analysis of land use, housing, demographic trends, community facilities, wildlife and vegetation, soil characteristics, air quality, transportation, land ownership, education, and recreation and open space. It must be recognized, therefore, that the findings and recommendations of the District's report serve as only one input into development of land use and comprehensive plans for the western C-9 basin. The other resource concerns delineated above, in addition to the District's hydrological analysis, are being addressed by the South Florida Regional Planning Council, with subsequent input to those local governments having land use planning jurisdiction in the western

C-9 basin.

The objective of this study, therefore, is to develop water management criteria and land use guidelines which are consistent with the hydrological limitations of the study area. In view of the fact that intensive water quality management studies are being conducted as part of the Broward and Dade County Section 208 Programs, water quality considerations are not given the same degree of analysis as storm water disposal limitations and water supply capability. The approach, instead, is to delineate existing water quality conditions and to recommend currently recognized storm water treatment mechanisms for implementation. Application of these mechanisms is considered to be an interim solution until said mechanisms are supplanted by actions taken subsequent to the Section 208 Plans. The storm water treatment mechanisms are included in the recommended land and water management criteria, delineated in Part V of this report.

II. INTRODUCTION

A. Regional & Local Features

The C-9 basin area lies on both sides of the Broward-Dade County line from the Intracoastal Waterway westward to Conservation Area 3B, Levee 33. The actual surface water basin covers an area of approximately 90 square miles, as shown on Figure 1.

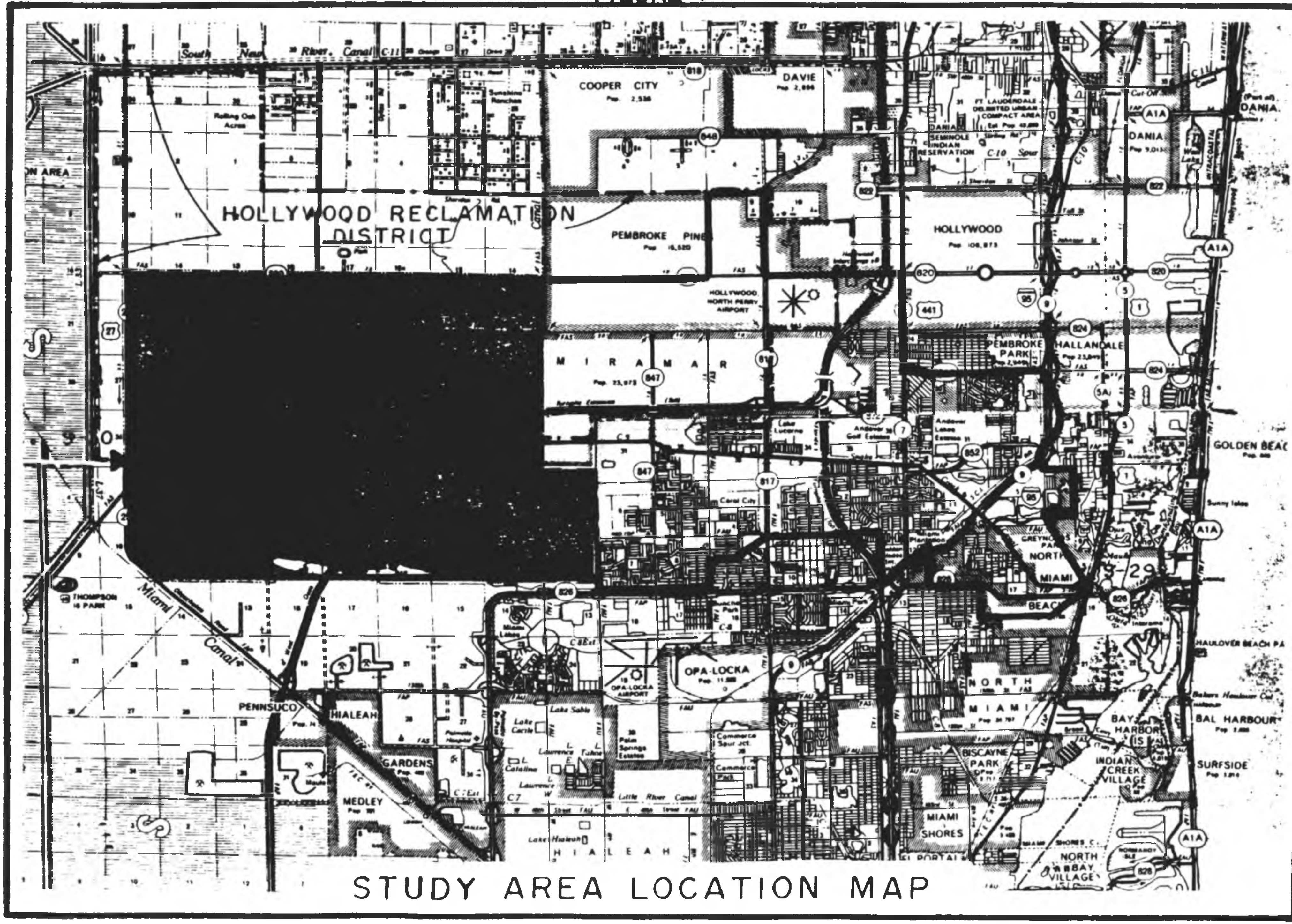
The basin area is nearly divided evenly east and west by the greater "Area A" and "Area B" of the Central and Southern Florida Flood Control Project. The eastern sub-basin is characterized by the presence of the coastal ridge. In the ridge area elevations of 10 to 15 feet* are common. A small portion of the eastern sub-basin is lower than 5 feet near the western end. The coastal ridge area can be generally characterized as sandy with some organic deposits, particularly in low area

In the eastern sub-basin 40% of the land area is occupied by residential use. Commercial, industrial, institutional and transportation use an additional 15% for a total of 55% fully developed with 27% impervious cover. The remainder of the eastern area is open and primarily undeveloped or in low agricultural use, mostly in the relatively low land near the western boundary of the area.

The western sub-basin, which is of primary interest in this study, lies west of the coastal ridge and east of the conservation area levee. The entire area is very low with an average elevation of about 4 feet. The natural land surface varies from about elevation 3 to elevation 6. The majority of the soils consist of less than 2 feet of muck overlying limerock. Significant subsidence has occurred in the twenty years since the Corps Project formulation and where undisturbed, will undoubtedly continue until the land surface is very close to the limerock.

* Henceforth, all elevations refer to feet above U.S.C. & G.S. mean sea level datum.

FIGURE 1



STUDY AREA LOCATION MAP

Because of the low elevations and poor drainage characteristics of the western sub-basin, development has not been significant. The existing residential area is only about 4%. The total impervious cover is 3% of the area. Limerock quarries are about 2% of the area. The balance of the area is low agriculture and undeveloped lands.

The climate of the area is semi-tropical and consistent with the rest of the Southeast Florida region. Average annual precipitation is approximately 60 inches. Average annual and extreme short term event rainfall depths are greatest along the coast and decline at points inland. About two-thirds of the average rainfall occurs in the four months of June through September.

B. Hydraulic Facilities

The existing main channel, C-9, was constructed under the Central and Southern Florida Flood Control Project. Sections 1, 2 and 3 form the main channel from the Intracoastal Waterway outlet to Red Road, which is the limit of "Area A" or the eastern sub-basin. The channel was designed and constructed to provide conveyance for estimated Standard Project Flood (SPF) inflows in the basin east of Red Road. Sections 4 and 5 were added later and brought the main channel from Red Road westward to Levee 33 and Structure S-30 on the east side of U.S. Highway 27. These two additional sections through the western sub-basin were not designed to provide any flood protection during the peak SPF condition. They were designed to facilitate drainage after capacity becomes available in the eastern reaches, and thus reduce the duration of inundation of the western sub-basin. The additional construction was also considered to provide for better control of stages at S-29, the salinity control structure at the basin outlet.

Structure S-29 is in continuous operation year round for the purpose of salinity control and for regulating water levels throughout the basin. There are four 22 feet wide vertical-lift gates designed to pass the eastern sub-basin SPF peak discharge of 4780 cfs. One gate is automatically regulated for headwater stage control. The operational controls are set to attempt to maintain a headwater stage of 2.0 feet throughout the year.

Structure S-30 at the west end of the canal consists of four 72-inch CMP culverts. For flood control considerations they are always closed, and are designed to be opened only during periods of drought when water is needed downstream for recharge and salinity control.

The secondary drainage system in the western sub-basin consists of ditches and shallow canals in north-south alignment at most section lines or one mile intervals. Connections to C-9 have been limited to a maximum discharge capacity in the west of 3/4 inch of water per day over the area served by the tributary system. In the eastern sub-basin inflows are unlimited because of the degree of protection provided by design.

Downstream of S-29, the main channel of C-9 has been rerouted through Maule Lake since that body of water has been cut open to the Intracoastal Waterway. This path avoids about two miles of the meandering Oleta River through mangrove marsh. With this route, for hydraulic computation purposes, the flow profile tailwater begins immediately upstream of S-29.

Channel cross-sectional surveying was done in 1975 by the Corps in C-9 and the secondary drainage system in Broward County in connection with the HUD flood hazard study. The cross-sections of C-9 generally have the conveyance called for in the original design. There are some constrictions at bridge crossings which are designed for nominal head losses.

Proposals for providing hydraulic facilities to permit development in "Area B" have been put forth in a series of "Survey-Review Reports" by the Corps of Engineers in 1957, 1958 and 1961. Stage construction was an essential feature of all plans. Those portions of "Area B" at highest elevations, specifically the Tamiami Canal area, would be initiated first. This would mean that improvements in the study area, with the lowest elevations, would be constructed last.

The 1957 plan considered a backpumping station at the western end of C-9 in Levee 33 with a capacity of 14,400 cubic feet per second (cfs), enough for total daily removal of the 30 year frequency rainfall and seepage from Conservation Area 3B. The necessary conveyance for this discharge required a channel dug to elevation -28.0 feet msl. At that time the estimated cost for the pumping station was about $14 million. The 1958 "Survey-Review" concluded that there was essentially no justification for Federal participation in the pumping plan.

The last "Survey-Review Report" for this area in 1961 recommended a combination of backpumping to Conservation Area 3B and fill and gravity drainage to the east for 30 year design storm protection. At the western end of C-9, a 2950 cfs pumping station was required for a removal rate of 2 inches of water per day. There would be inundation over much of the area for a duration of five days or less. That plan called for non-Federal cost sharing in primary project works of about 50%. No implementation or Congressional authorization of these Survey-Review Reports has been made.

The U.S. Geological Survey recently completed a study, using a groundwater analog model, for the placement of a secondary control structure at or near the sub-basin divide. The objective of the study was to evaluate the effectiveness of such a structure on groundwater conservation. The findings were that within the constraints of the model design, the secondary control effects were minimal. Further discussion of the structure is found in a subsequent section of this report.

III. ANALYSIS

A. Water Quantity Data

The primary input into most hydrologic systems is precipitation. For this study two rainfall stations were selected for a frequency study of rainfall depth. Miami WDO AP and Hialeah are National Oceanic and Atmospheric Administration (NOAA) stations with 35 and 34 years, respectively, of continuous daily rainfall record. They are both within 15 miles of the study area and, most importantly, are situated inland a distance about equal to that of the middle of the study area basin, avoiding coastal extremes.

For each station maximum events were selected from each year of record to form an annual series of one-day, two-day, three-day, four-day, five-day and one-month events. The annual series for each event at each station was then run through a log-Pearson Type-III frequency analysis and results for the two station averages are presented as Table 1. As expected, rainfall depths are similar for both stations and are very close at five-day and one-month levels. Therefore, the two stations' frequency-depth curves were averaged for use in the study basin. The resulting one-day maximum precipitation, 13.16 inches, agrees with the 100-year, 24-hour rainfall contour map presented in Technical Paper No. 40, U.S. Weather Bureau. For this reason there should not be too much concern that the local rainfall depth for five-day maximum at the 100-year frequency level is about 13% lower than the Corps of Engineers value for the more general "Greater Miami Area". The log-Pearson five-day maximum depths are greater than the values which log normal or Gumbel distributions yield from the actual data, although Gumbel gives a greater one-day maximum for the same data.

Only one rainfall station with a useful period of record has been actually sited within the study basin. Highly variable rainfall occurs across short distances in coastal South Florida. To overcome this variability, a Thiessen network of

TABLE 1

DESIGN RAINFALL DEPTHS (INCHES)

DAY	10-YEAR	25-YEAR	100-YEAR	SPF
1	0.33	0.24	0.13	0.20
2	0.60	0.90	1.51	1.89
3	7.26	9.37	13.16	16.45
4	1.96	2.40	3.23	4.04
5	0.49	0.43	0.17	0.21
Total	10.64	13.34	18.20	22.79

rainfall stations adjacent to the basin was constructed. Five stations were used for the network, although only three carry almost 95% of the total weight.

At the in-basin station, Jo-Jo Ranch, only nine years of record (1961-1969) are available. Therefore, it is only within this period and coincident period of discharge record that a rainfall-runoff relationship can be determined.

Discharge and stage records have been published by the U.S. Geological Survey (USGS) for S-29 (since 1959), 67th Ave. (since 1962) and S-30 (1963-1967). Discharges at S-30 after 1967 have been calculated from daily stage readings and culvert rating curves. The mean daily discharges, which are published, are computed by a stage and deflection meter relationship. By periodic streamgaging with a current meter, the accuracy of the deflection meter recordings can be estimated. For about half of the years involved, the USGS rates the accuracy as poor, or greater than 15% in error. The rest are rated fair, or less than 15% error. This error must be recognized and considered in those computations which use discharge records.

There are, or have been, five or six well recorders maintained by the USGS in the area of the western sub-basin. Because of the high transmissivity of the aquifer and the very active groundwater to surface water exchange, the observation well network is too sparse for a meaningful quantitative analysis. In this study of surface water hydrology the wells are used to estimate average groundwater stages throughout the sub-basin for conditions antecedent to project storms. A general direction of gradient can be seen from the hydrographs published by the USGS but interpolation of contours between well locations, many miles apart with ditches and canals in between is not accurate.

For this study, Central and Southern Florida Flood Control District (FCD) survey teams ran elevation lines over the entire forty-three square mile area comprising the western sub-basin. A copy of the elevation point map is given as Figure 2. Though over three hundred points were leveled to the nearest tenth of a foot, there are not enough points to construct a meaningful contour map in such flat terrain. By assuming that each leveled point is representative of an equal share of the area, it is possible to draw an "S" curve of elevation distribution as shown in Figure 3. With the large number of points taken on a uniform grid such a distribution should be acceptably accurate for purposes of this study. The average elevation of the land is much lower than was previously thought, the mean being about 4.1 feet, and the median about 4.0 feet. This is the most significant factor in behavior of the western sub-basin during flood conditions, both major and minor.

The surface water boundaries of the basin are well defined in the western sub-basin by roadways on three sides and the Golden Glades Canal and Levee on the south side. S-30 has only occasionally been opened for recharge purposes.

A somewhat temporary situation exists with a Hollywood Reclamation District pumping station of 50,000 GPM capacity in the C-11 basin being permitted to discharge southward down the Flamingo Road canal across Hollywood Blvd. This discharge enters C-9 immediately upstream of the 67th Ave. gaging point but does not actually enter the western sub-basin. Through unregulated connections in the Dade County secondary system there is flow southward into the C-8 system through at least two points. This water leaves C-9 immediately upstream of the 67th Ave. gaging point because of the gradient between the regulated stages of C-9 and C-8. Observations in the dry season indicate an estimated 50 cfs flow is occurring.

The groundwater boundaries are not well defined. There is apparently a very large flow seeping from the conservation area across U.S. Highway 27 into the northwestern corner of the study basin. This seepage accounts for the base flow in C-9 most of the year. This base flow averages 250 cfs during the rainy season when groundwater stages are highest, however, seepage in this basin has a more important role in water conservation than in extreme flood behavior. In perspective the flood water stored in the west at peak stage is greater than 42,000 acre-feet or equal to the highest estimated net seepage rates for about 85 days.

All of the above extra-basin influences will have little effect that cannot be accounted for in the flood performance routing of the basin. The Hollywood Reclamation District pumping station can be assumed to be operating at full capacity for the duration of flooding.

FIGURE 2

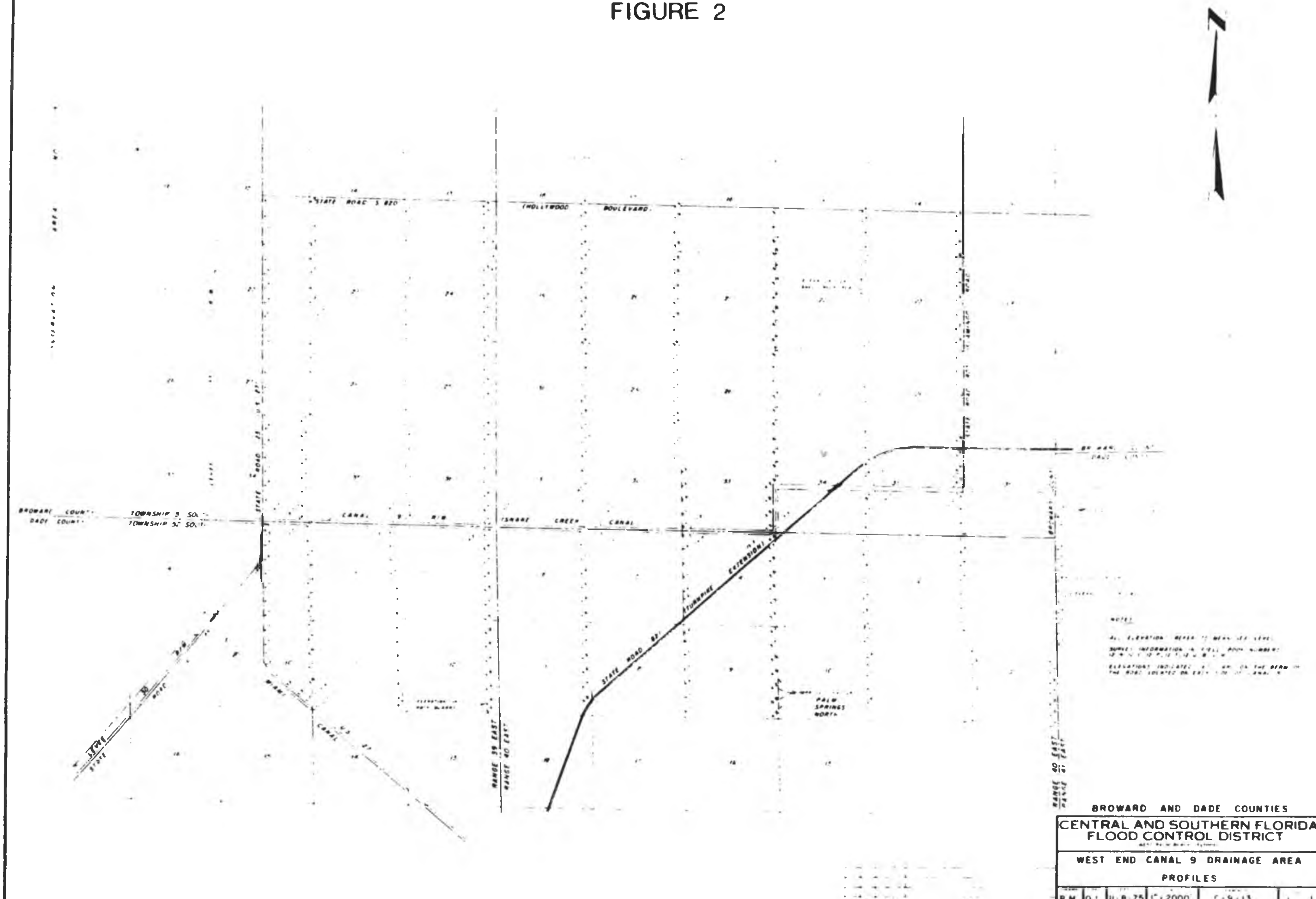
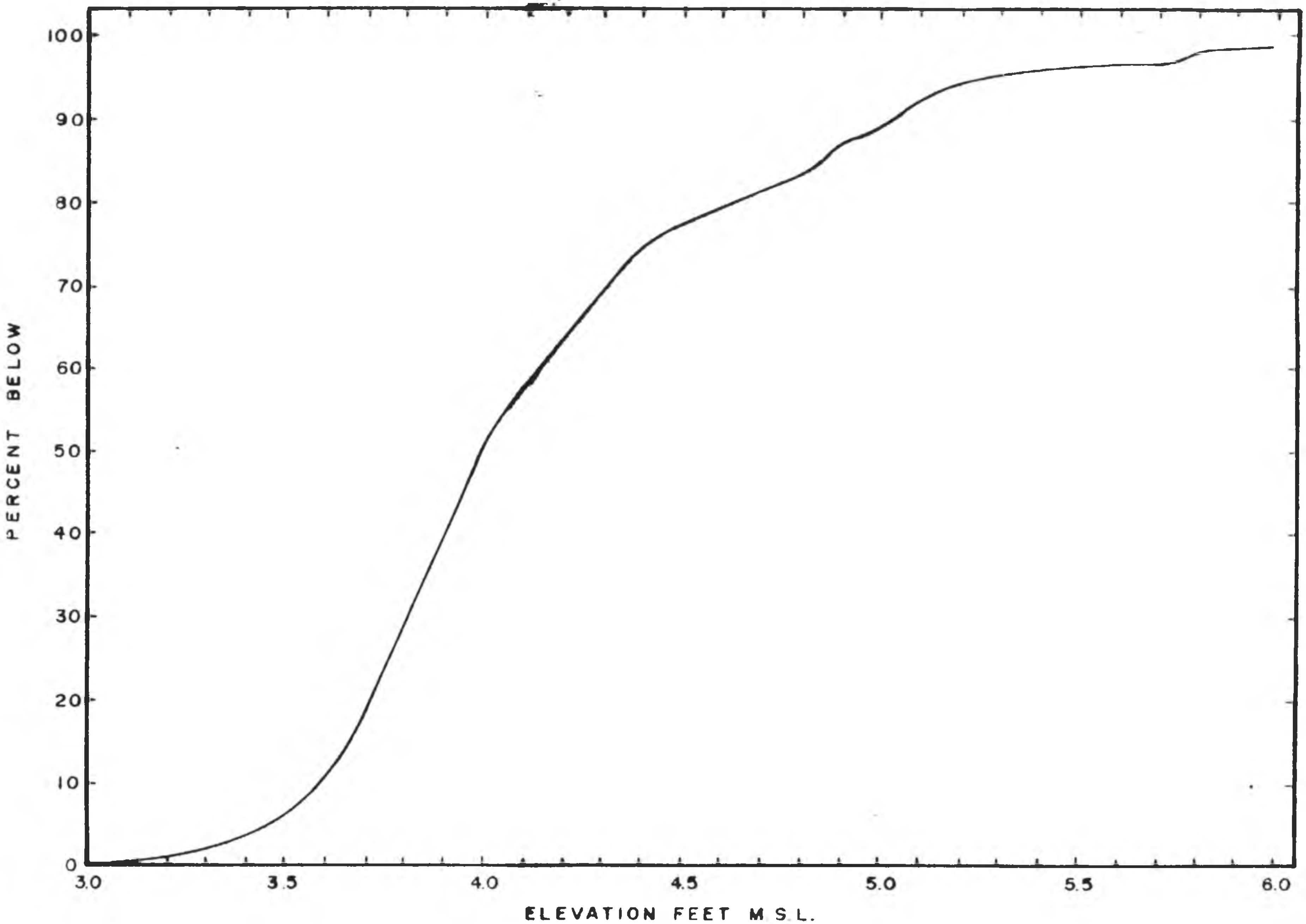

STATE ROAD S 820
HOLLYWOOD BOULEVARD
CANAL
SNAKE CREEK CANAL
TURNPIKE EXTENSION
STATE ROAD
PALM SPRINGS NORTH
RANGE 39 EAST
RANGE 40 EAST
BROWARD COUNTY
DADE COUNTY
TOWNSHIP 51 SO.
NOTES
BROWARD AND DADE COUNTIES
CENTRAL AND SOUTHERN FLORIDA FLOOD CONTROL DISTRICT
WEST END CANAL 9 DRAINAGE AREA
PROFILES
R M
O L
1"=2000

FIGURE 3



GROUND SURFACE

B. Water Quality Conditions

1. Surface Water

Tables 13 through 23 exhibit surface water quality data for the Western C-9 (Snake Creek Canal) basin (reference Appendix D, Water Quality Data). Tables 13 to 15 present data assembled by the U.S.G.S. since 1963. Tables 16 to 18 are data collected by the Broward County Environmental Quality Control Board. Tables 19 to 23 contain water quality data assembled by McGinnes and Associates in the preparation of an ADA for the proposed Country Lakes Development. Figure 19 presents a history of BOD, total and fecal coliforms, and dissolved oxygen since 1970. This figure also gives a diurnal profile of D.O. for June 1975. The data are from C-9 at station 43 and were published by the U.S.G.S. Sampling site locations are shown on Figure 18.

The nutrient levels as presented, in general, show little enrichment. The varying levels of both nitrogen and phosphorus indicate their sources are stochastic, i.e., they are probably of non-point source origin. There is little development in this basin; the pollutant levels, especially towards the west, therefore reflect natural background concentrations for a poorly drained muckland area. An examination of potential nutrient sources indicates that direct rainfall could be a significant source. FCD staff, in an investigation of Lake Okeechobee, reported rainfall to have concentrations of total N= 0.73 mg/l and total P= 0.038 mg/l. In another study by Joyner the total nitrogen and phosphorus were reported to be 0.90 and 0.056 mg/l, respectively. Comparing the nutrient concentrations as measured in C-9 to the aforementioned rainfall concentrations illustrates lower phosphorus and slightly higher nitrogen concentrations for the majority of measurements. These measurements indicate there is a source (or sources) of nitrogen in the basin, and also indicate, by the fluctuations in concentrations below those found in rainfall, that there is some nutrient uptake in the canal system. The nitrogen increases probably emanate from the decomposition of the muck soils.

In examining Tables 19 through 23 the following observations about the surface waters can be made: (1) phosphorus concentrations are highest at stations L-1, reflecting surface discharge from the low density pasture lands in the north; (2) phosphorus concentrations at station C-9 (west) are the lowest, reflecting the lack of phosphorus sources in the western sections of the basin; and (3) nitrogen concentrations are lowest in the western section of the basin, and increase to the east.

The U.S.G.S. data, as presented in Figure 19, show consistently low BOD's, below 2.0 mg/l. The total coliform counts average about 1000/100 ml, with variations from 100 to 10,000 counts/100 ml. The fecal coliforms are generally in the range of 10 to 100/100 ml, but show more variation than total coliforms. The dissolved oxygen levels are consistently low, but have been increasing since 1970. The inherently low D.O. concentrations are indicative of the groundwater/surface water interchange, while the increasing D.O. suggests less groundwater movement into the surface waters during this period.

Discharge of treated domestic wastewater cannot be considered as a major source of water quality degradation in the western reach of the C-9 basin. There are four wastewater treatment plants in the western C-9 basin, two in Dade and two in Broward County. In Broward County, Heritage City Utilities operates a 0.15 MGD (design flow) sewage treatment plant at the northwest corner of the basin. This facility discharges to a self contained lake, and should therefore present no major surface water degradation problems. Hollywood Lakes Country Club facility operates at high BOD and SS removal efficiencies and also has low flows (20,000 gpd ADF). This plant discharges to a secondary canal and presents no significant water pollutant problems to C-9. Data for a third plant, the Haven Lakes Estates Mobile Home Park, is also presented because of its close proximity to the study area. This sewage treatment plant is the only facility that discharges directly into the C-9 canal that could have effects on the western C-9 basin. In the dry season, or in periods of low flow this discharge could affect water quality upstream of the discharge. In Dade County the Country Club of Miami operates a contact stabilization, 1 MGD design flow facility. Its discharge is used for spray irrigation of the golf course. Palm Springs North has a contact stabilization plus a pressure filter operation. Its design flow is 0.75 MGD, and discharges to the Peter Pike Canal (77th Ave. Canal, just west of the Palmetto Expressway). None of these facilities should constitute a major source of water quality degradation in the C-9 canal, although the discharges of the Hollywood Lakes Country Club and Palm Springs North may locally cause enrichment in the secondary canals.

The assimilative capability of the canals in this area are limited because of the inherently low DO levels, the deep narrow canal contours, slow movement, and low turbulence.

2. Groundwater

Some groundwater quality data is given in Table 25. The locations of these sample sites are given in Figure 18. Sampling sites G-219, S-1490, and S-1495 are USGS well locations. Groundwater quality data is also included from the wellfield for the Country Club of Miami. The groundwater has low turbidity, high color, and high hardness. The color is probably a result of the contact of the groundwater with decaying organic matter in the muck layers and from vegetable and organic extracts, and hardness is high, probably due to the interaction of groundwater with the limestone formations. Chloride concentrations are seen to increase at G-219 with depth from 19 ppm at 32 feet to 408 ppm at 198 ft.

C. Existing Performance

1. Extreme Events

The main channel, C-9, has actually no bottom slope from S-30 to S-29. Its bottom varies from point to point between elevation (-)12 feet to (-) 14 feet. The land surface in the east has appreciable slopes because of the coastal ridge. In the west no effective slopes exist. Some difficulty should therefore be expecte in moving water from point of impact in the western sub-basin to the main channel and then through the main channel eastward. Physically, the only way in which water can move from one point to another is by stage difference. The water must literally be stacked up on one end to push water out the other end.

The lowness of the entire western sub-basin becomes very important in this procedure. Large volumes of water will accumulate before there is sufficient stage differential for any significant movement. This stage difference must equal the friction loss of western sub-basin outflows through the eleven miles of channel in the eastern sub-basin to S-29 plus that friction loss of any inflows which are occurring in the east. In this respect, flow out of the western basin is not so much limited by channel conveyance as by the overall lowness of the area. In fact, the western area has an affinity for storm water generated in the east during extreme events.

For all computations the tailwater at S-29 was assumed to be at 2.0 feet. The choice of this stage is in recognition of the so-called "Spring Tide" and off-shore winds which occur in the fall. Examination of stage records at stations in the Intracoastal Waterway show daily maximums of about 3.0 feet and minimums of about 1.0 feet during these tides. The duration of such extremes can run for days. Since the time of the year in which these high tides occur coincides with the wet season and greatest expectation of unusual storms, to use a lesser tailwater condition would be unreasonable. This event, which occurs annually, gives some insight to conditions of poor drainage which have been often observed in the wet season. Half of the land in the west is only one foot or less above a fairly common (3.0 foot) high tide more than ten miles away. In such a state, gravity drainage is almost nonexistent in the west.

The scenario of events during and after an extreme storm can now be described. The daily routing for extreme storms of 100-year and SPF frequency is presented in the Appendix. Behavior in the eastern sub-basin is somewhat normal with some land slope and a good deal of imperviousness. Runoff response is reasonably quick. The main channel in the east is filled with runoff before water in the west can reach the channel. Stages in the eastern reaches of the channel increase to a point above the ponded water in the western sub-basin. This means that during the peak runoff condition in the east, one day after peak rainfall, there must be some diversion of water from the eastern area westward into the western sub-basin. This occurs for one day in the 100-year routing and for two days in the SPF (100-year rainfall depths increased 25%) routing. From that point on, the eastern area is clearing, permitting discharge out of the west to begin.

The western area is totally inundated by the end of the maximum one-day rainfall and reaches a peak stage with additional rainfall two days after the maximum rainfall. These peak stages are computed as 5.4 feet and 5.9 feet for the 100-year and SPF events. For the 100 year routing, the duration of flooding back down to antecedent stages of 3.0 feet is about 30 days with 20 days to reach 4.0 feet. The SPF routing will require about 5 additional days of discharge to reach the same elevations. The maximum outflow for the 100-year routing is 1510 cfs or 1.3 inches and occurs 11 days after the rainfall peak.

The backflow condition into the west does not significantly increase stage in that area, since it would be distributed over the entire 27,600 acre area which is inundated at the time. For the 100 year event, approximately 1200 cfs flows backward during that unique day, which is less than an average of 0.1 foot over the western area. For SPF the backward flow is about 1800 cfs one day and 1000 cfs the next day for a maximum stage increase of slightly more than 0.1 foot average.

The backflow does, however, dramatically reduce stages through the main channel in the eastern area since that water is entirely within the banks. At the time of peak discharge through S-29, stages in the channel are 5.2 feet and 5.7 feet for the 100 year and SPF respectively, at a point about two miles eastward from 67th Ave. This location temporarily becomes the equilibrium point about which water flows east to one side and west to the other. One or two days later when peak pond stage is reached in the west, all flow in the eastern channel will be eastward and from the pond stages of 5.4 feet and 5.9 feet, 100 year and SPF, respectively, canal stages will decrease eastward until the tailwater of 2.0 feet is reached at S-29.

If the respective county fill criteria have been met for all buildings there should be no stages in the basin east or west, even at SPF levels, for which those buildings are endangered. This is true only for existing conditions, and specifically for existing development in the west. The western sub-basin acts as a detention area, holding all of its storm water while the eastern area is clearing. Extensive fill in this area should be termed encroachment and will have the effect of displacing flood water storage, creating higher ponding stages and forcing water eastward creating higher stages in the east. If allowed to proceed to near total areal development, the result in the eastern area would be stages which could be damaging to existing development at the 100-year frequency level.

2. Behavior During Non-Flood Periods

One of the most notable hydrologic features of the study basin is the high rate of groundwater seepage into the canal system. This is apparent in the form of base flow at the 67th Ave. and S-29 gage points. Seepage is greatest during the wet season when the base flow for the average year is estimated at 250 cfs passing 67th Ave.

There is a high statistical correlation between monthly discharge out of the basin and stage in Conservation Area 3B. For this reason and from a study of groundwater contours it can be concluded that groundwater moves from the Conservation Area across U.S. Highway 27 and into the basin in a general east-southeast direction. The groundwater is intercepted partially by the main channel but principally by the secondary drainage ditches in a north-south alignment north of C-9. Even shallow ditches are suprisingly efficient collectors in this alignment and this area of high transmissivity. The term "transmissivity" refers to the ability of a porous groundwater aquifer to transmit water. USGS estimates for the area are 4-6 MGD per foot of head.

In the channel reaches near S-30 there is a constant inflow from the north through the secondary system even throughout the dry season. This overdrainage should not be necessary to maintain downstream stages, as S-30 is available for that purpose. A four foot deep ditch between Hollywood Blvd. and C-9, one half mile east of U.S. Highway 27, has been observed discharging 30 to 50 cfs during the dry season with no antecedent rainfall. For such a shallow penetration to produce so much water the aquifer must be very unconfined and the transmissivity extremely high. Construction of a groundwater model about the area suggests that the transmissivity could be 8-10 MGD per foot of head.

On an annual basis, discharges for almost all years exceed precipitation. For some of the years the basin discharge approaches 100 inches. In contrast, for the idealized South Florida basin, the typical runoff from 60 inches of rainfall would be only about 15 inches. Since S-29 gate operations have been automated not enough annual records are available to determine the gate's effectiveness for water conservation, though automation should cause some added efficiency.

Drainage of normal rainfall during the wet season is severely restricted because of the topography in the west. Low elevations and lack of slope prevent much normal runoff that should occur. High groundwater levels prevent much downward percolation of water. Consequently, in many areas water accumulates in ponds for several weeks. The hydrologic cycle approaches that of an evaporation pan - precipitation in and evapotranspiration out.

The seepage observed in the study area represents a mechanism of such complexity that this report will not attempt to fully analyze in quantitative terms the seepage or its consequential problem of water conservation. It is recognized that very large volumes of fresh water are being lost to the sea each year from the basin outlet. Solutions to the problem will not be derived easily and, insomuch as the problem is not unique to the study area, solutions will evolve from systems-type studies of the entire region being carried forward in the District's "Water Use & Water Supply Development" for the Lower East Coast planning area.

As has been previously stated, the seepage problem has no appreciable effect on major flooding. And if seepage could be completely eliminated there would still be drainage difficulties after common rainfall events.

D. Performance with Development

Practically the entire western sub-basin is inundated with a 100-year flood. Therefore, any amount of fill placed on existing ground will displace an almost equivalent volume of flood water and consequently increase flood stage. Since existing flood levels are below any stages predicted by previous studies, there should be some acceptable level of development encroachment before damages begin to accrue. Such encroachment would permit some development in the western area without endangering existing development in the east.

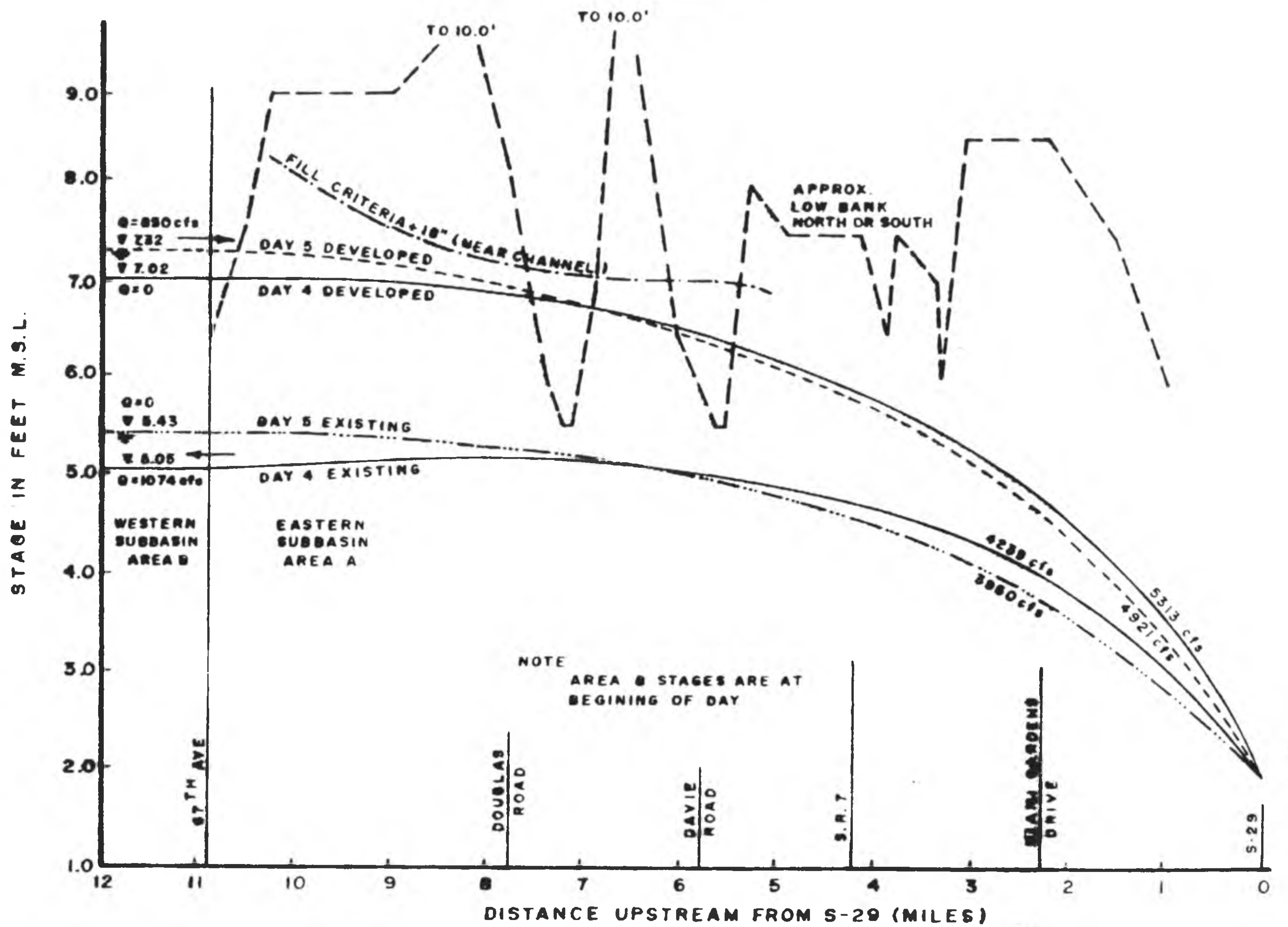
It would be very difficult to determine an exact stage-damage relationship along C-9 through the eastern reaches. The Corps' design backwater profile reaches a stage of 7.05 feet at the 67th Ave. divide. In the current analysis, the 100-year profile at peak runoff condition in the east would reach 7.02 feet at the same point if no water were permitted to flow westward at that point. This is, therefore, a limiting condition on encroachment. Since both counties' fill criteria is between 6.0 feet and 6.5 feet at this end of the eastern sub-basin there should be no major damage to buildings set 18 inches above fill criteria with such a water surface profile. If fill is permitted in the west to an extent such that the pond stage is the same as the backwater profile from the east there will be no flow across the boundary on the critical day.

Before a level of encroachment is determined, some allowance for fill must be made for areas already under development or presently zoned for development. Five sections in Dade County and one-half section in Broward were thus considered for a total of 3520 acres or 12.75% of the western sub-basin. For these areas it is assumed that total fill over existing ground will be the ultimate result. The effect will be a modification of the stage-storage curve. With the new curve it was determined that displacement fill of 1.95 feet over the entire remaining area would create a pond stage of 7.02 feet at the time of peak runoff in the east. This is at the end of the third day of rainfall, the maximum one-day event, and comparable to a stage of 5.05 feet with existing conditions. Further increase in stage occurs on the fourth day as additional rainfall exceeds discharge. The peak stage would be 7.3 feet and comparable to 5.4 feet with existing development. These stages and backwater curve profiles through the east are presented in Figure 4.

The 1.95 foot, or 2.0 foot of fill volume will be the basis of fill criteria developed further hereinafter. The manner of placement of this fill has an effect on the flood elevations for lesser events, such as those of 25-year and 10-year frequencies. If the fill is placed in the lowest areas possible, the pool stages will be at maximum elevations. Most probably the fill would be placed on the highest land first to utilize the greatest portion of the area, and consequently, some of the fill would have no effect on the lesser flood elevations. Multiple ownership in the basin and piecewise development would prevent an optimum fill placement, so that the probable configuration would be somewhere between the two extremes.

The lowest placement would result in a perfectly flat basin surface at elevation 6.5 feet. Routing the 10-year and 25-year rainfall events results in peak stages of 6.5 feet and 6.8 feet, respectively. Fill placement on the highest ground would fill all land above 3.5 feet and result in peak stages of about 5.5 feet and 6.3 feet for the 10-year and 25-year routing. Since there is no way of determining how the basin will finally be developed the most conservative set of elevations should be used.

FIGURE 4



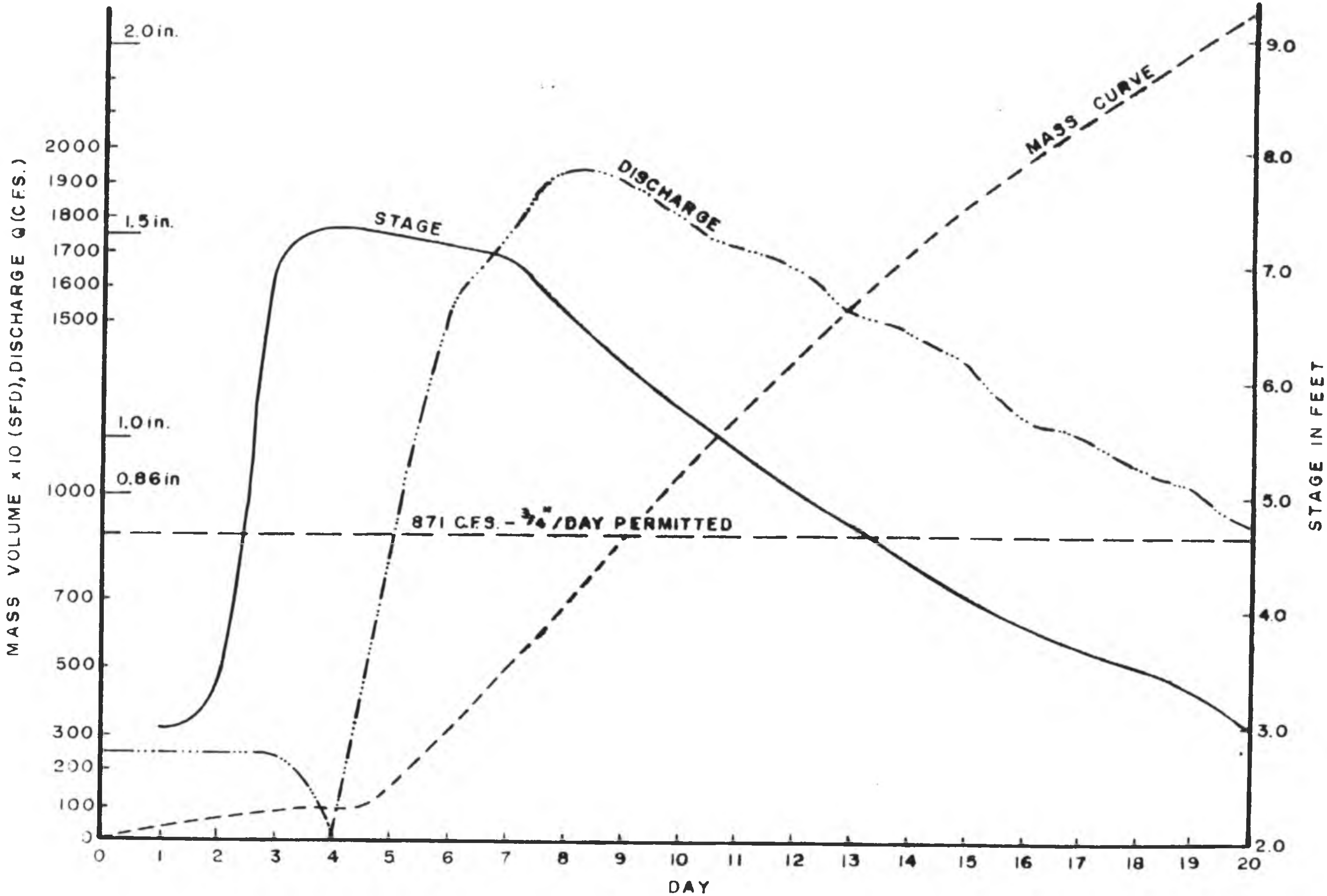
100-YEAR BACKWATER PROFILES

The foregoing computations of fill effects assume that all fill is imported from outside the basin. However, in the above case most of the fill will probably be obtained on-site for economic reasons. The effect will be that there is no encroachment created by removal of soil between antecedent groundwater levels and land surface. Since these effects are indeterminable with existing data, assuming groundwater at land surface will introduce a slight safety factor in the overall analysis. Digging below the water table and placing the borrow on the land surface will actually decrease storage and therefore must be considered the same as importing fill from outside the basin.

Routing for duration of flooding for the 100-year event shows about 15 days of pool stage above elevation 4.0 feet and twenty days to reach 3.0 feet. The maximum net outflow from the western sub-basin occurs five days after the rainfall peak and is 1960 cfs or 1.69 inches. This reduction in duration from the existing condition is due to the increased stages with fill encroachment and the resulting increases in outflow from the area. The daily stage and discharge routing is graphed in Figure 5.

The western sub-basin behavior with allowable fill encroachment is very close to the original design. At least half of the fill will effectively replace the subsidence which has occurred in twenty-five years. Contour maps of the area at that time indicate an average elevation of about 5.0 feet, one foot higher than the existing condition.

FIGURE 5



DEVELOPED 100-YEAR WESTERN SUBBASIN ROUTING

IV. CONCLUSIONS

A. Technical

1. Flood Stages

In this analysis it is shown that the western sub-basin does not behave appreciably different in concept from that of the original Corps' project report. Though subsidence has altered the relative stages of water between the two sub-basins, the basic mechanism of downstream channel control is still valid. Additional subsidence will exaggerate the backflow conditions and depress stages of flood in the west and channel stages in the east even further. It is conceivable that this subsidence could continue until either solid material or dry season groundwater levels are reached. Under such conditions, backflow will occur from the east after even minor storms and drainage from unfilled land in the west will be virtually nonexistent.

In relating peak flood stages to existing building elevations, the western sub-basin has greater than 100-year flood protection. Channel conveyance is more than adequate in the east for the estimated 100-year discharge. It should be recognized, however, that local flooding will probably occur in parts of the eastern sub-basin even though the primary drainage system is apparently more than adequate. The local flooding would be caused by the inability of the secondary and tertiary drainage system to deliver storm runoff to C-9 after rainfall events. It is not probable to expect that secondary and tertiary drainage can be improved to the point of significantly modifying the rainfall runoff relation because of excessive costs associated with construction in highly developed areas.

This report then recognizes that local flooding can occur in the eastern sub-basin but that the primary system has capacity for relief since the receiving waters in the main channel are lower than most of the land. Areas with good drainage access to C-9 will have minimal flooding problems.

Peak flood stages in the western sub-basin are summarized in Table 2. These stages are determined from a routing (Appendix B) which assumes a flat pool over the area. This is valid for the more extreme events when there is little water movement out of the basin. However, for the 10-year and 25-year events there will be some slope or contour of the water surface as there is appreciable overland flow of shallow depth. As explained previously, the stages are based on the worst or limiting cases of fill placement and are, therefore, quite conservative. These stages more than compensate for a water surface slope in the lesser floods.

TABLE 2

WESTERN BASIN FLOOD STAGES

Frequency	Existing Elevation	Developed
SPF	5.9	7.8 (2)
100-yr	5.4	7.3
25-yr	- (1)	6.8
10-yr	- (1)	6.5

(1) not routed
(2) estimated

If the fill limitations stated in this report are utilized for guidance of future development in the western sub-basin, there should be no endangerment to existing development in the east or west at the 100-year level because of the fill. County fill criteria in the west dictates that all residential buildings have floor pad elevations of at least 8.0 feet. The proposed 100-year stage is below this level.

2. Water Conservation Limitations

For conservation of fresh water it is desirable that construction and development not increase groundwater seepage rates into the primary drainage system. The vehicle for this seepage would be the rock quarries, borrow pits, and secondary drainage systems. Unfortunately, at the present, it is not possible to accurately predict the quantitative effects on seepage of such earthwork. It is easy to see that a long quarry pit aligned normal to the groundwater contours would increase seepage by providing a path of no resistance along its length. But a draft of restrictive criteria for these features is technically indefensible at this time.

Excessive drainage is a basin feature related to the high seepage rates. The secondary drainage system of ditches and small canals effectively penetrates the water table even at the dry season levels. This is occurring most significantly in the northwest quadrant of the sub-basin area. It seems desirable to not drain groundwater below at least the dry season water table levels. An examination of USGS annual dry season water table contour maps indicates that drainage should cease when the water table drops to elevation 2.0. The upstream stage at S-29 is also regulated for an optimum control stage of elevation 2.0 through the entire year.

To reinforce this control the inverts of culverts discharging directly to C-9 should be fixed at a minimum elevation of 2.0. For open channel connections to C-9 weir controls should also be installed with a minimum crest elevation of 2.0. With these measures, dry season discharge into the canal system should be very low. There should also be some reduction in wet season discharges without impeding storm runoff.

For maximum water conservation benefits the invert elevations of such controls should be set higher than elevation 2.0 or have a crest which could be varied throughout the year. Having fixed inverts higher than elevation 2.0 could restrict stormwater drainage in some areas and variable controls at many locations would present a problem that could be beyond the management and enforcement capability of the governing jurisdictions. Therefore, a 2.5 foot control elevation for direct connections to C-9 is a compromise solution between the two distinct problems. In addition, for indirect connections to C-9, the control elevation is established at six inches below average existing ground elevation for the subject parcel.

As previously noted, provision of a secondary control structure in the main channel near the sub-basin divide was considered. The structure would be intended to conserve water by holding higher than existing stages in the entire western area. Additionally some flood protection benefit was envisioned as a consequence of providing the capability of detaining storm runoff from the west.

It has been shown in this study that storm runoff detention via a secondary control structure is not necessary since it occurs naturally. The topographic survey shows that raising the water table over the area by even two feet would bring the water up above the existing ground surface in much of the area. Consequently, there are practical limitations of raising the water table elevation throughout the basin, since the basin is undergoing transition from a rural to an urban character, thus creating the potential of adversely impacting existing land use activities in the lower areas.

3. Water Quality

As indicated earlier herein, surface water quality in the western C-9 basin, in general, shows little degradation. Dissolved oxygen levels are a problem, however, probably due to the high degree of interaction between surface and ground waters. In addition, the information analyzed in conjunction with consideration of current regulatory agencies policies and criteria, indicates that non-point derived pollution will continue to be the major potential source of water quality degradation in the western C-9 basin. Filling of the area, in conjunction with more intensive development, may have significant effects on water quality in the basin, due to storm water runoff. These effects have not been quantified in this report, since several on-going planning efforts are addressing the non-point source problem in this area, including primarily the Section 208 studies in Broward and Dade counties, and the Lower Florida Basin water quality management planning effort of the Department of Environmental Regulation. In light of these facts, the approach taken in this report is to recommend currently recognized surface water quality control procedures and mechanisms for implementation during the interim period period prior to implementation of the Section 208 plans. These recommendations are included in the water management criteria delineated in section V.

In terms of groundwater quality, excavation of deep lakes for fill, withdrawal of limerock for commercial use, storm water detention/retention, or other purposes, has been identified as a potential source of degradation. However, sufficient data to document the impacts of deep excavation (more than fifteen feet, generally), if any, on the groundwater quality of the area, does not currently exist. In addition, this particular issue was considered to be beyond the scope of this study. The approach taken, instead, is to recommend control of surface water inflows to the excavations through application of the surface water management criteria in section V. These criteria will be applicable until such time as the Section 208 plans are implemented, or until sufficient documentation regarding impacts on groundwater quality is made available through other efforts.

B. Administrative

1. Surface Water Management

An effective stormwater management plan must ensure that future changes within the basin do not result in a reduced level of flood protection. In essence, provisions must be incorporated into this plan to prevent an increase in the established regulatory flood level.

To meet this objective two primary considerations must be addressed:

a. Stormwater discharges under ultimate development conditions must be restricted to those values which do not alter the existing hydrologic behavior of the basin.

b. The basin must be protected from encroachment which would reduce storage capacity.

The purpose of this section of the plan is to consider the first of these considerations while the latter is discussed under the flood plain management section.

DISCUSSION OF ISSUES

The regulatory flood levels and associated flood profiles were determined in this study. The methodology used in these determinations, which utilized existing and committed land uses in calculating runoff,was discussed in Section III.C.

It is apparent that future development beyond that considered as existing or committed in this study has the potential to alter the basin hydrology significantly. Given the potential for such additional development, procedures are needed to assure that the allowable runoff limits are not exceeded.

These allowable runoff constraints raise several issues involving other than purely technical considerations. Since it is apparent that storage facilities are required to handle any runoff in excess of the specified allowable discharge quantities, several alternative methods for providing retention capacity are available; herein lie the policy issues that must be examined.

ISSUE NO. 1

Should new development be required to provide detention/retention for the design storm, while prior development is "grandfathered", or should the burdens of providing retention be shared among both existing (including or excluding committed) development and future development?

CONSIDERATIONS:

1. In cases where adequate detention/retention can be incorporated into a drainage system at the design stage, it is more economical to provide detention/retention capacity for new development than existing development.

2. From the viewpoint of a new developer, it is not equitable to place the full burden of detention/retention on only new development, since the new developer is placed at an economic disadvantage relative to prior development.

3. For committed, but as yet unconstructed developments, could some uses be reasonably required to meet higher levels of runoff control through redesign?

4. In most cases, limitation of runoff from existing development could be economically implemented only through provision of downstream

detention/retention capacity.

5. In terms of ease of implementation, restriction of runoff (provision of detention/retention) can be ranked in the following order of increasing difficulty: future uncommitted development; committed, but unconstructed development; existing development.

6. From a water quality perspective, provision of detention/retention facilities for both future and existing development would result in a greater improvement in the quality of stormwater runoff.

ISSUE NO. 2

Given that detention/retention of excess stormwater will be needed in the basin,should detention/retention capability be provided on a site-by-site basis, on an overall sub-basin basis, or through some intermediate alternative?

CONSIDERATIONS:

1. The provision of detention/retention capability on a site-by-site basis, i.e., for each individual development has several associated disadvantages:

    a. Difficulties in designing appropriate facilities for small parcels and some large parcels with inappropriate natural features.

    b. Higher unit costs of construction, operation and maintenance.

    c. Less efficient and less reliable operation and maintenance.

    d. Greater susceptibility to locally intense rainfall occurrences, thus creating system design problems.

2. Associated with individual on-site retention facilities are the

following advantages:

a. Ease and timeliness of implementation, concurrent with development.

b. Allocation of costs not to the general public, but to the individual developer and subsequent residents who benefit.

3. By providing detention/retention capability on a sub-basin basis, a different series of advantages and disadvantages require consideration:

Advantages

a. Greater flexibility in selecting appropriate and economical retention sites.

b. Enhanced economies of scale in construction, operation and maintenance.

c. Greater efficiency and reliability in operation and maintenance.

Disadvantages

a. Necessity to institutionalize with the associated problem of long lead time, funding and cooperation.

b. Allocation of costs to the public rather than the private sector. This objection could be ameliorated by formation of a special purpose taxing District.

4. An additional significant consideration involves the degree of institutionalization desired for the basin. The requirements upon local government under the individual on-site detention/retention approach would require an expansion of the existing types of activities such as review of plans and specifications, inspection, etc., due to the

increased number of such facilities. In addition, monitoring and enforcement activities would increase.

The sub-basin approach to providing detention/retention capacity would require a much greater level of governmental involvement, including sub-basin-wide water management planning, governmental acquisition of retention areas, and designation of entities to carry out the functional responsibilities of design, construction. operation, maintenance, and financing.

In summary, then, based on the assumption that existing and committed development would be permitted to discharge surface water under the criteria applicable at the time of commitment, the two basic alternatives for providing runoff retention are:

1. On-site retention facilities for each individual development.
2. Retention facilities on a sub-basin or planning unit basis.

For each alternative, runoff would be limited in accordance with the recommended criteria in section V.A.

2. Flood Plain Management

There are many ways in which flood plains can be managed. These range from allowing open space developments, to allowing fully developed flood plains accompanied by enlarged channels to compensate for flood plain uses which reduce existing conveyance capacity. The most satisfactory use varies with each community depending upon flood hazards, land-use needs, cost of management alternatives, and so forth. Comprehensive community-wide planning can define land-use needs and the availability of flood-prone and non-flood-prone sites to satisfy these needs.

Ultimately, the ability and resolve to manage development in the western C-9 Basin so as to minimize flood damage potential rests with local governments through the powers granted to them by the State. Under Florida Statutes, local authority to regulate the use of flood plains in order to reduce losses to life and property in the interest of the public welfare is clearly a 'police power.'

Traditionally, there have been three basic policy objectives in regulating land use in flood plains under this power: (1) protection of those living in the flood plain; (2) protection of others from the consequences of development in the flood plain with its resulting obstruction of flood flow; (3) protection of the entire community from individual choices of land use which require subsequent public expenditures for public works and disaster relief. Underlying these concerns is another general policy objective that is gaining recognition and support: planning and regulating land use in a manner more consistent with the inherent constraints of the natural environment.

It is significant that court cases throughout the country have shown that when conscientiously applied, regulatory controls over land use in flood plains protect the rights of private individuals, and do not constitute a taking of property. This is primarily because good flood plain regulations do not prohibit all use of land, but rather establish development criteria consistent with nature's demands for conveying flood flows and the community's land use needs.

Proposing land use criteria and designing zoning regulations for a flood plain, like all other planning, depends upon local conditions; but it also depends upon the flood data available and whether or not the regulations are to be combined with other regulations. The final section of this report outlines recommended land use criteria for flood plain regulation in the western C-9 basin. These criteria are designed to meet the standards for land use controls established for the Federal Flood-Insurance Program, with the qualification that federal criteria, like provisions of model ordinances, must sometimes be adapted to local circumstances.

Usually, a two-zone approach is found suitable for developing land use criteria where engineering studies are available on flood hazard areas. The two-zone approach consists of developing a set of land use criteria for each of two zones designated as the floodway (that portion of the flood plain consisting of the stream channel and overbank areas, and capable of conveying and/or storing a selected flood discharge and keeping it within specified heights and velocities) and the flood fringe (that portion of the flood plain beyond the limits of the floodway). This method is most appropriate in urban areas in which property values are high and the demand for land is great, long range land use plans are available and alternatives for flood plain management have been carefully considered. However, as evidenced by the data and analysis presented earlier herein, the entire western C-9 basin could be considered as a "floodway". Thus, the recommendations for permissible land uses and land use intensity delineated in section V are based on the western C-9 basin being considered as a "floodway". In general, this approach delineates the criteria necessary to maintain the existing hydrologic behavior of the basin under specified flood events by making provisions to allow unobstructed flood flows (to the extent practicable) and allowing only the amount of fill which will not cause an increase in the regulatory flood level.

V. Recommendations

A. Surface Water Management

1. Recommended Approach

After consideration of the factors discussed earlier herein, the following approach to regulation of surface water management is recommended.

a. Require on-site detention/retention facilities in accordance with the water management criteria delineated later in this section.

b. Encourage use of joint retention facilities by new developments, if feasible.

c. Conduct detailed studies on a sub-basin level to evaluate the feasibility of acquiring and using downstream facilities for community rather than individual development (on-site) detention/retention.

For each of phases a-c, planning and evaluation will necessarily have to proceed in accordance with the information and data provided in this report (or subsequent, more detailed information), and in accordance with the recommended surface water management criteria, as specified below.

2. Surface Water Management Criteria

Based on District staff experience in the Development of Regional Impact, Surface Water Management Permit, and other evaluation processes, a review of criteria being applied by the Dept. of Environmental Regulation and various local jurisdictions, and the findings of this study, the following criteria are recommended for the regulation of surface water management systems. These criteria should be incorporated into the applicable local ordinances, including subdivision regulations, planned unit development regulations, and others as deemed appropriate.

Definitions

a. "Master Surface Water Management Plan" means an engineering drawing and a written report outlining the primary and secondary drainage and storm water treatment facilities. The Plan shall indicate the method of drainage, existing water elevations, drainage structures, canals and ditches, the storm water treatment methods, necessary percolation and detention and management areas and any other pertinent information pertaining to the control and management of surface and ground water.

b. "Storm Water Treatment" means the natural, chemical, biological or physical process by which the quality of storm water may be controlled, and may include but not be necessarily limited to, the following: (a) water storage facilities, such as golf course lakes, real estate lakes, impoundments, dikes, and roof-top storage; (b) detention and filtration techniques, such as grassy swales, routing of runoff through vegetated areas (marshes, cypress hammocks, etc.), sedimentation basins, dams, step-down weirs, catchment basins, grates, screens, baffels, skimming devices, oil and grease separation devices, circulation and flushing mechanisms, use of dispersed or sheet flow in lieu of other transport mechanisms, street cleaning, chemical treatment, and other appurtenances and processes; and (c) mechanisms to control runoff resulting from construction activities, such as diking; turbidity control diapers; use of hay bales, mulching, seeding, sodding on cleared land areas; regrading to moderate slopes; minimize amount of land cleared at any one time; sedimentation traps; retention basins; and holding ponds. This definition does not normally include active treatment processes, requiring the consumption of electrical or mechanical energy such as those processes used in plants similar to water supply or wastewater treatment plants.

c. "Reviewing Jurisdiction": Any governmental entity having regulatory and/or jurisdictional authority with respect to drainage.improvement projects and water resources management within all or parts of the Western C-9 Basin.

General Requirements

a. Drainage: An adequate drainage system, including necessary ditches, canals, swales, percolation areas, detention/retention ponds, storm sewers, drain inlets, manholes, headwalls, endwalls, culverts, bridges and other appurtenances shall be required for the positive drainage and control of storm and groundwater. The drainage system shall also provide for off-site runoff affecting the proposed developed area. For design purposes, the 100-year, 25-year, and 10-year flood frequency elevations are established as 7.3 feet, 6.8 feet, and 6.5 feet msl, respectively.

b. Storm Water Quality Control: Storm water treatment facilities shall be required to maximize storm water quality by providing for on-site percolation and/or detention or any other appropriate treatment technique for storm water.

Design Requirements

a. The surface water management facilities shall be designed to handle the three year storm based on the current Florida Department of Transportation intensity curve applicable to the Canal 9 Basin. The system shall provide for the drainage of lots, streets, roads and other public areas including surface waters which drain into or through the property proposed for development. The design shall provide for surface water management of adjacent contributory areas.

b. Gravity drainage systems shall be designed to retain the first one inch of runoff and the runoff from a three year, one hour storm. In addition, the average detention time for runoff from a twenty-five year, twenty-four hour storm shall be five hours.

c. For systems designed to be pumped from fully diked areas, discharge shall be limited to three-fourths of an inch per twenty-four hours, or the criteria in b, whichever is more stringent. In addition, no pumping shall be permitted when canal stages at pump tailwater exceed the twenty-five year peak elevation of 6.8 feet msl.

d. All direct connections to C-9 shall be designed to prevent lowering of the groundwater table below elevation 2.5 feet msl. All indirect connections to C-9 shall be designed to prevent lowering of the groundwater table by setting the culvert invert elevations or by installation of fixed weirs at an elevation six inches below average existing ground elevation for the subject parcel.

e. The runoff coefficients used in the design shall be those applicable after complete development has occurred and shall be calculated on sample areas of each type of ultimate use.

f. The surface water management system shall be designed for long life, low maintenance cost and ease of maintenance by normal maintenance methods. The minimum pipe used within a storm sewer system should be 15 inches in diameter. Maximum swale grades shall be limited to that grade which will produce water velocities below the threshold of erosion.

g. The surface water management system shall be designed using acceptable engineering principles with consideration being given to the protection of all future buildings from a one-in-one-hundred-year storm.

h. Rainfall runoff, surface and ground waters shall be managed to minimize degradation of water quality and discharge of nutrients, turbidity, debris, and other harmful substances, and maximize percolation and detention to promote the re-use of the resource.

i. Runoff from roads, parking lots, roofs and other impervious surfaces should be directed over areas where percolation into the soil can be accomplished prior to introduction into any storm sewer or other transport facility.

j. Runoff which must be carried directly into the closed storm sewer system without previously crossing percolation areas should be discharged to percolation areas prior to conveyance to on-site bodies of water, or off-site receiving waters in order to promote detention, disposition of silt and other particulates and the removal of nutrients or other undesirable constituents in the water prior to discharge from the development site.

k. Moderate berms should be constructed around the perimeter of excavations to promote seepage rather than direct discharge. A slope of 7:1 to a depth of 3 feet should be required around the perimeter of lakes and other excavations to promote improvement of runoff quality and for safety reasons.

l. Swales may be used in lieu of storm sewers to convey and collect surface waters.

m. Alternate methods or facilities which in the opinion of the reviewing jurisdiction are equal or superior to the requirements stated herein may be approved. Application for such approvals shall be accompanied by written data, calculations and analysis which show, by accepted engineering principles, that the alternate methods or facilities are equal or superior to those specified.

n. The design data of the surface water management system shall be submitted along with the Master Surface Water Management Plan in a report form prepared by the developer's engineer indicating the method of control of storm and ground water, including the method of drainage, existing water elevations, proposed design water elevations, drainage structures, canals, ditches, and other pertinent information pertaining to the system. The material submitted shall clearly indicate adherence to these conditions, and shall be furnished to the reviewing jurisdiction.

Surface Water Management System Maintenance and Operation. The following conditions shall apply:

a. The proposed development's surface water management system shall have an appropriate and functioning authority legally assigned the responsibility for operation and maintenance of the system in which jurisdictional authority is exercised. No integral parts of the surface water management system shall be without an appropriate and functioning authority for operation and maintenance. Appropriate authorities include but are not limited to, improvement districts, new community districts, drainage districts, cities, villages, towns, property owner's associations, cooperative associations, Broward County, and Dade County.

b. All major surface water management facilities such as swales, lakes, canals, and other detention/retention areas used for surface water management prior to discharge from the development shall be placed in water management tracts dedicated to the authority responsible for their maintenance.

c. Design solutions which require periodic maintenance shall only be used where an appropriate and functioning maintenance authority is in existence or will be established.

B. Flood Plain Management

In terms of permissible land uses, the following land uses have a low flood damage potential, and do not obstruct conveyance of flood waters. Thus, these uses could be permitted within the western C-9 basin to the extent that such uses are not prohibited by any other ordinance, and the applicable fill criteria, specified later herein, are adhered to.

1. Agricultural activities, including but not limited to, general farming, pasture, grazing, outdoor plant nurseries, horticulture, viticulture, truck-farming, sod-farming, and wild-crop harvesting.

2. Recreational activities, including but not limited to, private and public sports, both active and passive, such as golf courses, tennis courts, driving ranges, picnic grounds, hiking and riding areas, and wildlife and nature preserves.

3. Industrial and commercial activities, including but not limited to, loading areas, parking areas, airport landing strips and rock mining operations.

These permissible uses, in terms of flood control, should be taken into consideration in development of land use plans for the western C-9 basin. However, it must be recognized that water quality factors, particularly in regard to the agricultural activities, should be considered in the on-going planning efforts for this area, in addition to the urban resource constraints.

Regarding more intense land use activities, including residential development and other urban land uses requiring structural improvements, fill encroachment criteria are recommended which provide for 100-year flood protection to the eastern C-9 basin, with no backflow into the western C-9 basin. Allowance has been made for approximately five and one-half sections of land either already developed or assumed as committed for development, by assuming total filling in those areas. With the assumptions and considerations enumerated earlier in this report, the following criteria are recommended.

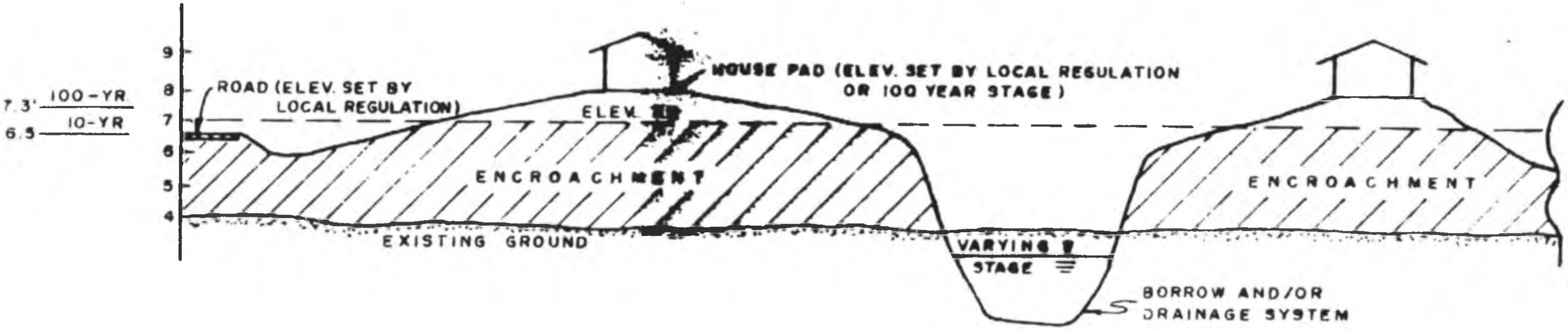
1. The volume encroached by development between average existing ground surface and elevation 7.0 feet msl shall not exceed 2.0 feet times the total area of the property.
2. For diked areas with on-site retention of runoff, the area diked shall not exceed the above encroachment volume divided by the difference between average existing ground elevation within the dike and elevation 5.75 feet msl. This will require all such projects on land of average elevation less than 3.75 feet msl to preserve some area outside of the dikes with no fill.
3. Structures shall be constructed such that the first floor and basement floor are above the regulatory flood protection elevations, as a minimum, as designated by local government. Fill should be at a point no lower than one (1) foot below the regulatory flood protection elevation for the particular area and should extend at such elevation at least fifteen (15) feet beyond the limits of any structure or building erected thereon. House pads should be at least six (6) inches above the regulatory flood level.

Typical development schemes using these criteria are depicted in Figure 6 for easy reference.

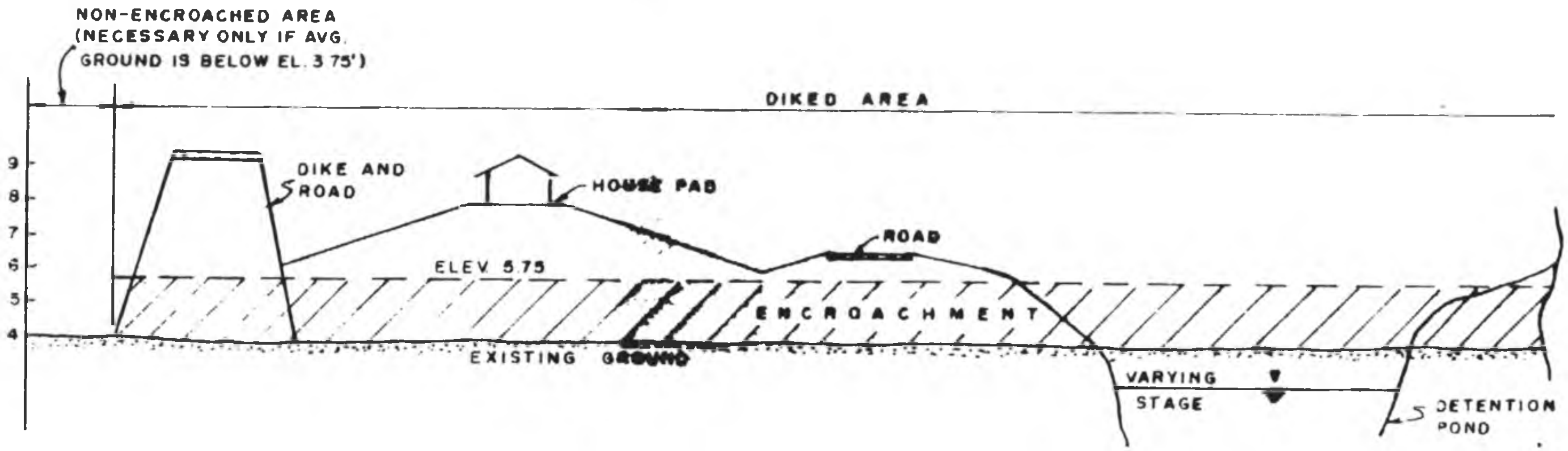
It is recommended that the above land management criteria be incorporated into the applicable local ordinances for implementation, including subdivision regulations, planned unit development regulations, and other ordinances as deemed appropriate. In addition, the procedures established to implement these criteria should also consider provisions for granting special permits and variances, provided that an acceptable engineering evaluation substantiates the need for the special permit or variance.

Not only should these criteria be included in the day-to-day management of land use in the western C-9 basin, but such criteria should also be used in development of short and long range land use and comprehensive plans for the area. However, caution must be exercised in applying the results and recommendations contained herein to areas outside the western C-9 basin.

FIGURE 6



GRAVITY DRAINAGE SYSTEM



PUMPED RETENTION SYSTEM

TYPICAL DEVELOPMENT SCHEMES

C. Summary

Based on experience in South Florida, the approach delineated in this report of using flood plain management criteria, in terms of permissable land uses and intensities, in conjunction with regulation of surface water management systems, offers a reasonable vehicle to ensure that the established regulatory flood level will not be elevated. Each development parcel will have its own engineering and planning challenges, and individual technical approaches may be required. It must be recognized that at times the results may be at the level of "technological break through" regarding such problems.

The success of this approach now rests in the hands of the local governments having jurisdiction in the western C-9 basin, in applying these criteria to their planning and regulatory activities in this area.

APPENDIX A

Eastern Area Runoff Determination

In attempting to arrive at rainfall-runoff relationship for the eastern sub-basin it became apparent that empirical techniques for runoff estimation are inadequate in the study basin. Even in the eastern half of the basin runoff response is measured in days instead of hours. Land cover data is not really useful when groundwater levels are at or near the surface since the effect is that the saturated area becomes impervious. SCS runoff methods were attempted but the results could not approach historical discharges.

For these reasons it is felt that a linear model or unit hydrograph based upon historical precipitation and discharge is a realistic method of estimating runoff for design events. The Thiessen network provides the rainfall average and the two discharge recording stations provide a net runoff measurement.

The Thiessen network employs five stations which are given in Table 3 with their respective weights. The weighting was determined with a District computer program employing a Monte Carlo sampling technique although the graphical method is probably just as good. It is seen that one station, Jo-Jo Ranch, dominates the network with 68% of the weight. This is a now discontinued station so that analysis of historical events is limited to the life of that station.

TABLE 3

THIESSEN NETWORK RAINFALL STATIONS

Station	Weight
Jo-Jo Ranch	68%
Pennsuco	18%
S-13	9%
S-9	4%
S-13A	1%
	100%

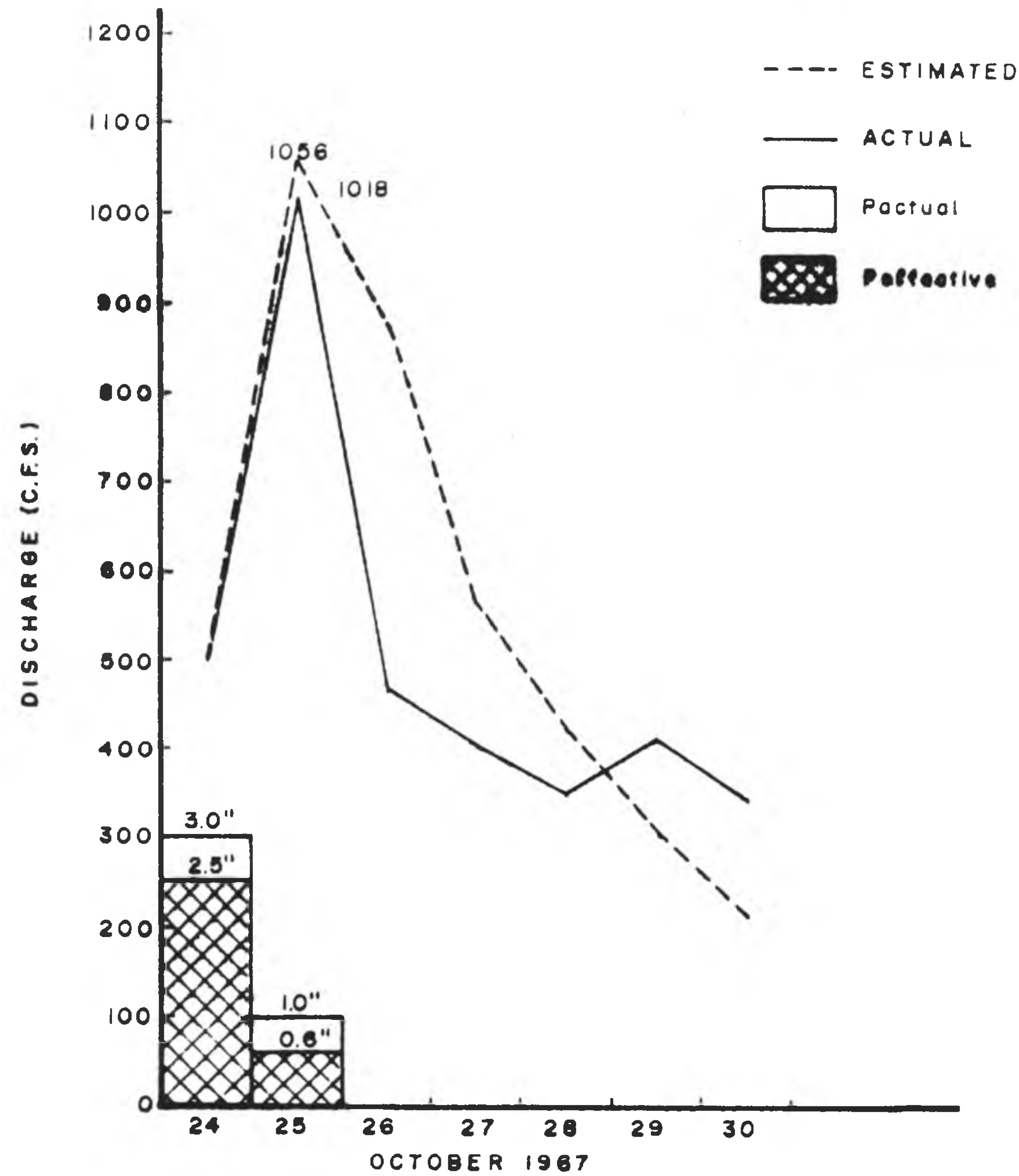
It is seen that even in the eastern sub-basin, which behaves very normally when compared to the west, the discharge hydrograph has a duration of about two weeks after a single day's rainfall. To find a dry antecedent period of such duration, a one-day rainfall of sufficient magnitude and areal uniformity, and a dry succeedent period is impossible during the wet season. Therefore, a dry season (May) event was selected for analysis. This would not normally be a good practice since hydrologic conditions are quite different for the two seasons. However, the resulting unit hydrograph works out well enough when tested with actual wet-season events.

The selected event, 3 May 1963, had a fairly uniform rainfall distributed over all of the Thiessen stations with the weighted average being 3.27 inches. The total discharge hydrograph at S-29 minus 67th Avenue flows and base flow is given in Figure 7. From the fifth to the twelth day the recession limb is estimated. A summation of the discharge volume over the drainage area indicated that an SCS curve number (CN) of 95 would yield the correct amount of runoff (2.71 inches) from the rainfall. The total hydrograph is divided by this effective rainfall to give the unit hydrograph shown in the Figure.

This unit hydrograph is applied to more complex wet-season rainfalls of two and three consecutive days in Figure 8 and 9. The peaks are within 10% of the actual measured discharges and the first seven days' volumes are within 12% of measured values. Both of these relative errors are within the accuracy of the field measurements. The predicted values are also greater than actual which suggests some factor of safety.

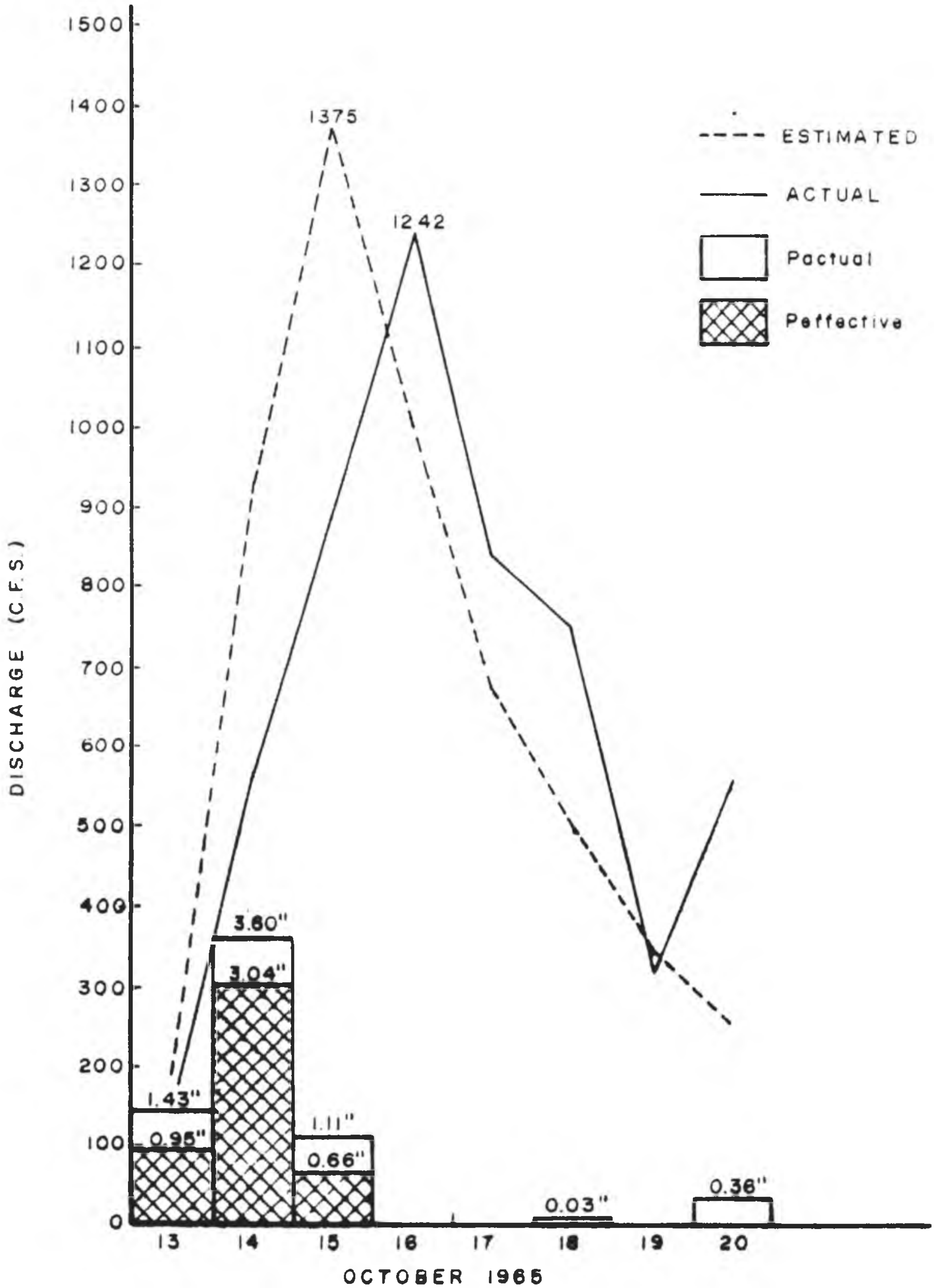
It should be stated that the foregoing derived unit hydrograph should not be considered as any definitive analysis of eastern sub-basin response or applied in any further design in that sub-basin. It is used here only as an estimator

FIGURE 8



NET DISCHARGE AT S-29

FIGURE 9



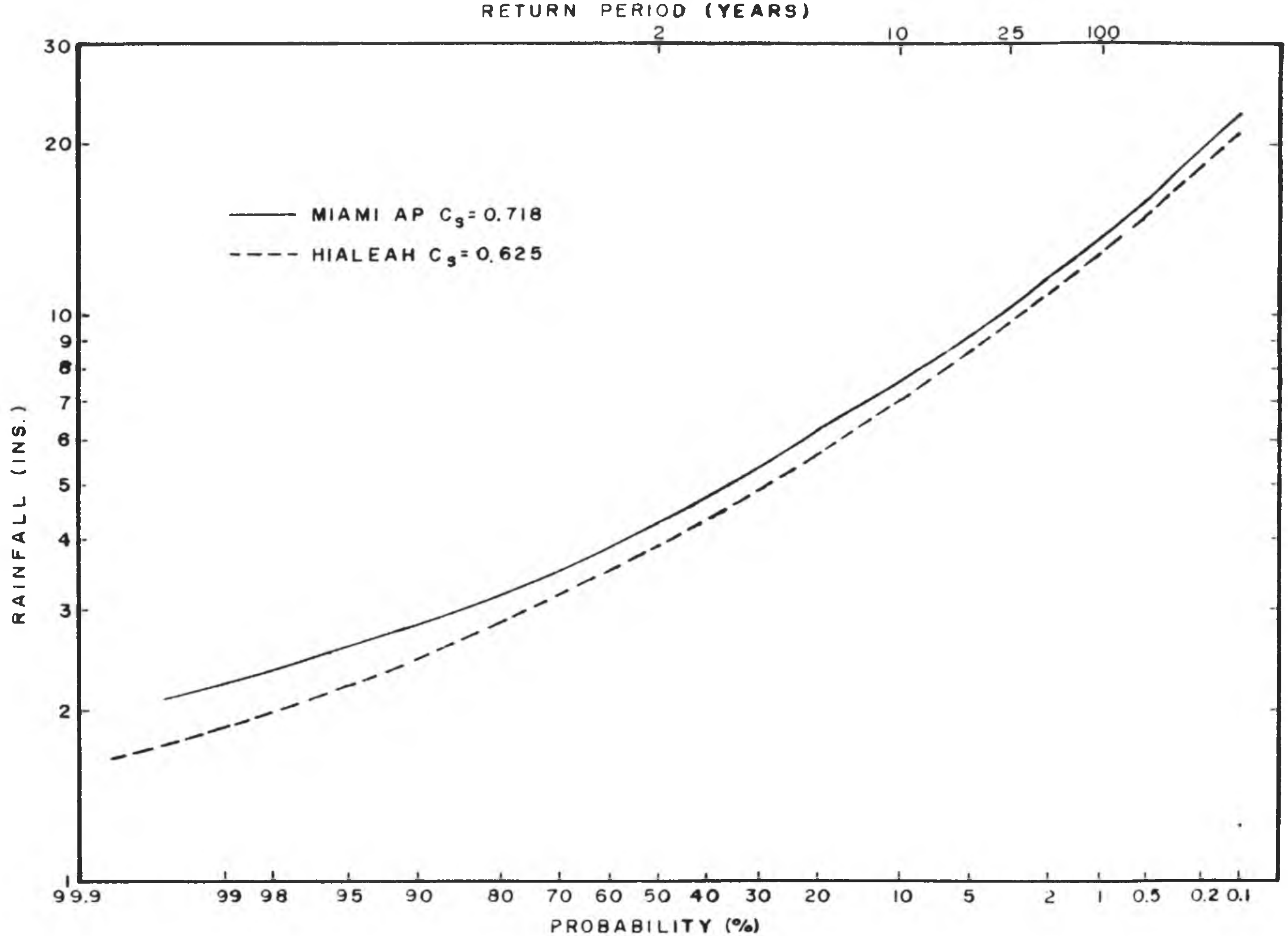
NET DISCHARGE AT S-29

for the peak stages in the east as they affect the western sub-basin. As the unit hydrograph reasonably imitates the historical peak runoff råtes, it serves well enough for this purpose. After the peak has past, eastern runoff conditions primarily affect the duration of flooding in the west. As seen in routings, inundation duration on the order of thirty days are expected in the west; an actual difference of five days in either direction is not critical in effect.

The rainfall depth-frequency curves as derived from the frequency analysis are presented in Figures 10 through 15.

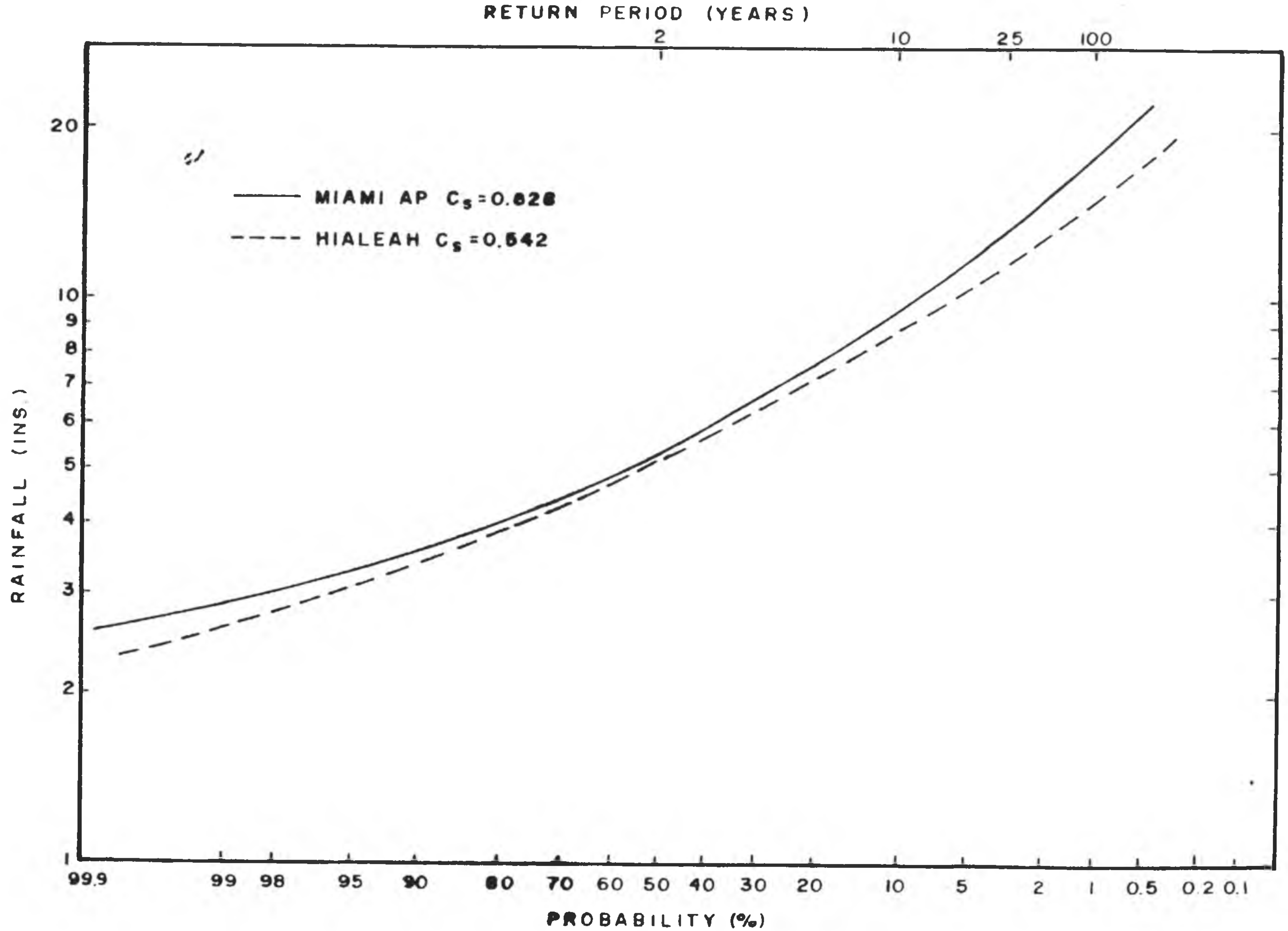
Rainfall depths for SPF, 100-year, 25-year, and 10-year 5-day events are applied to the unit hydrograph and presented in Tables 4 through 7. Base flow seepage and pumpage are included to give total daily discharge in the eastern sub-basin for each event.

FIGURE 10



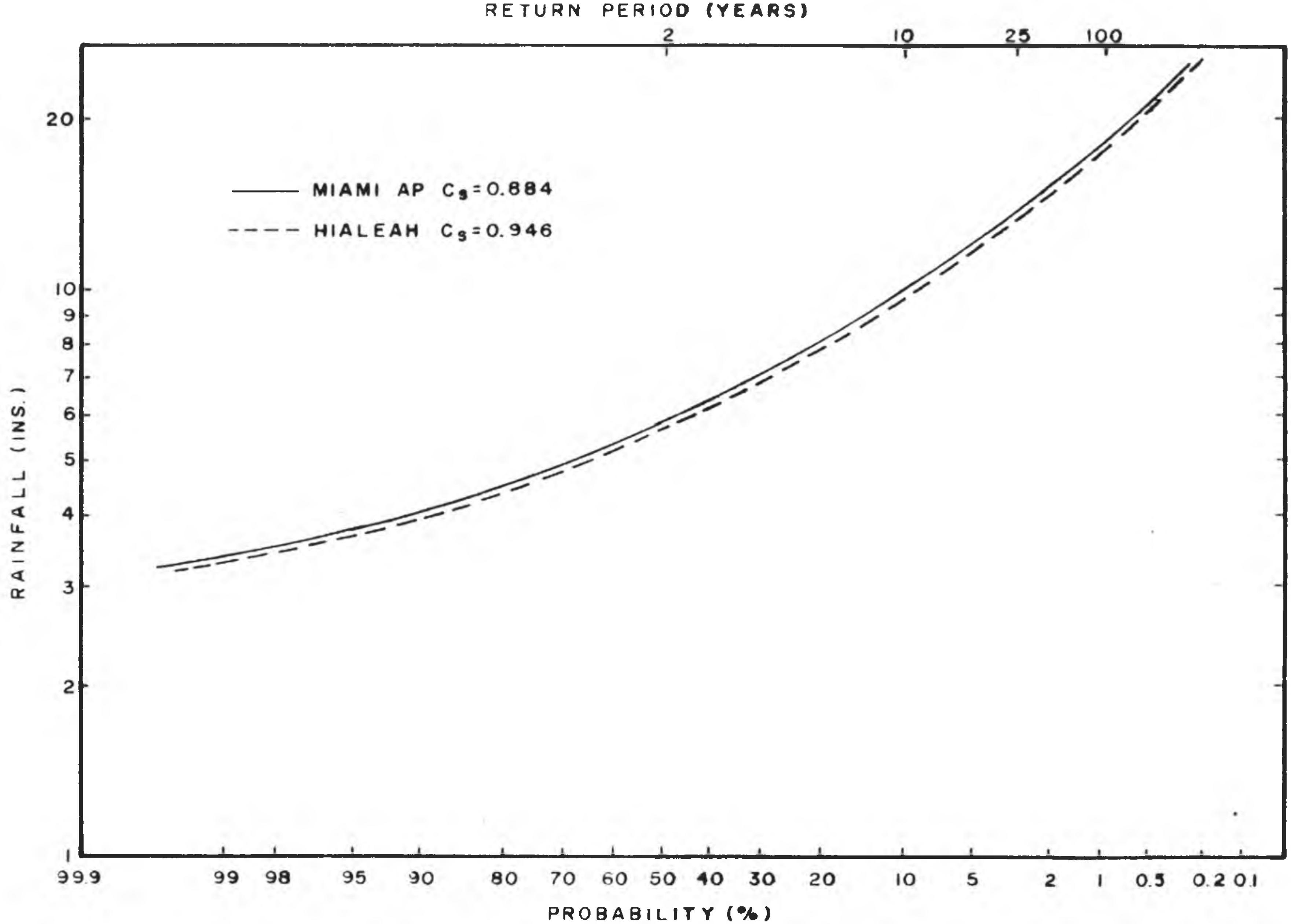
ONE-DAY MAXIMUM PRECIPITATION

FIGURE 11



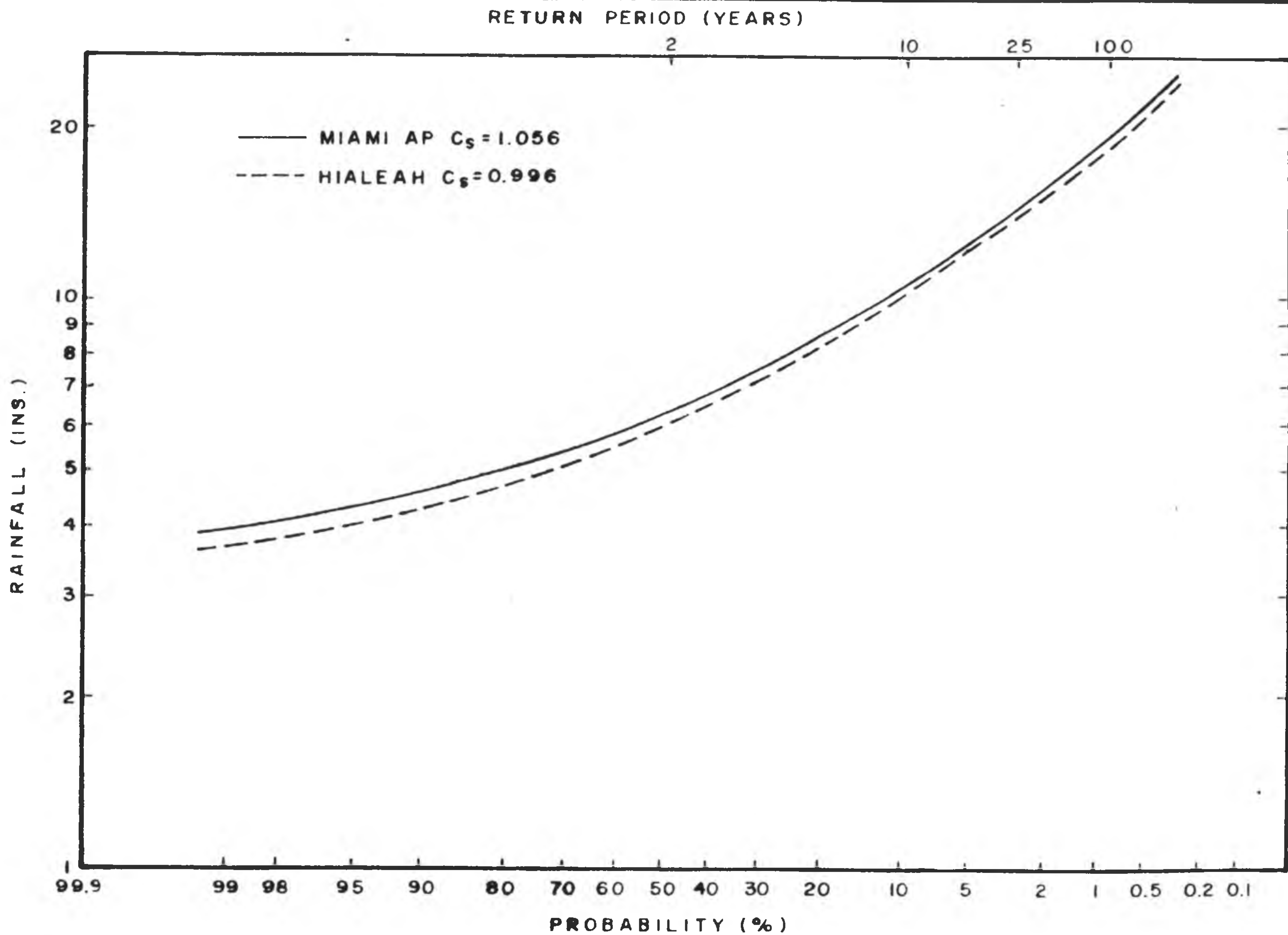
TWO-DAY MAXIMUM PRECIPITATION

FIGURE 12



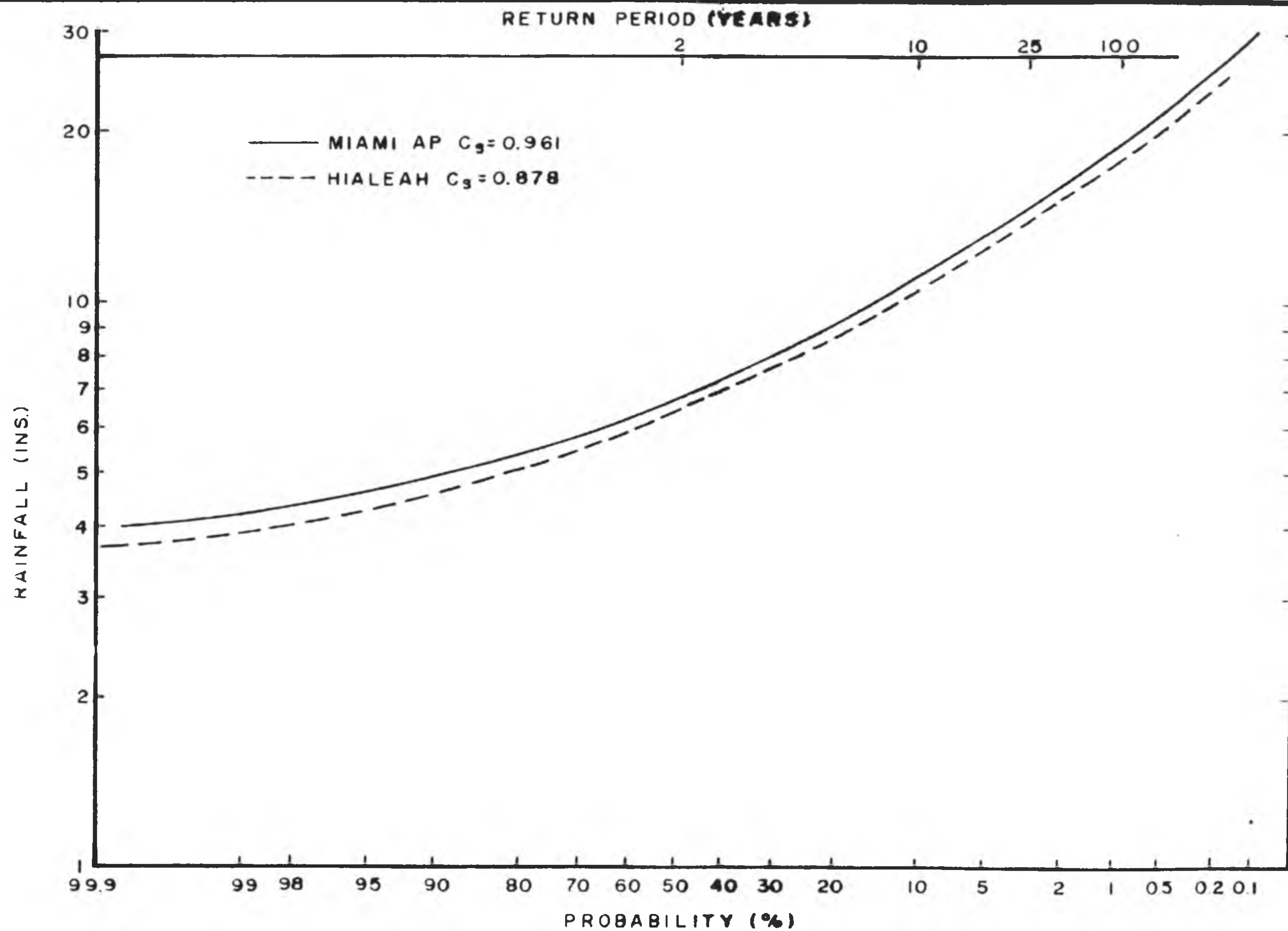
THREE-DAY MAXIMUM PRECIPITATION

FIGURE 13



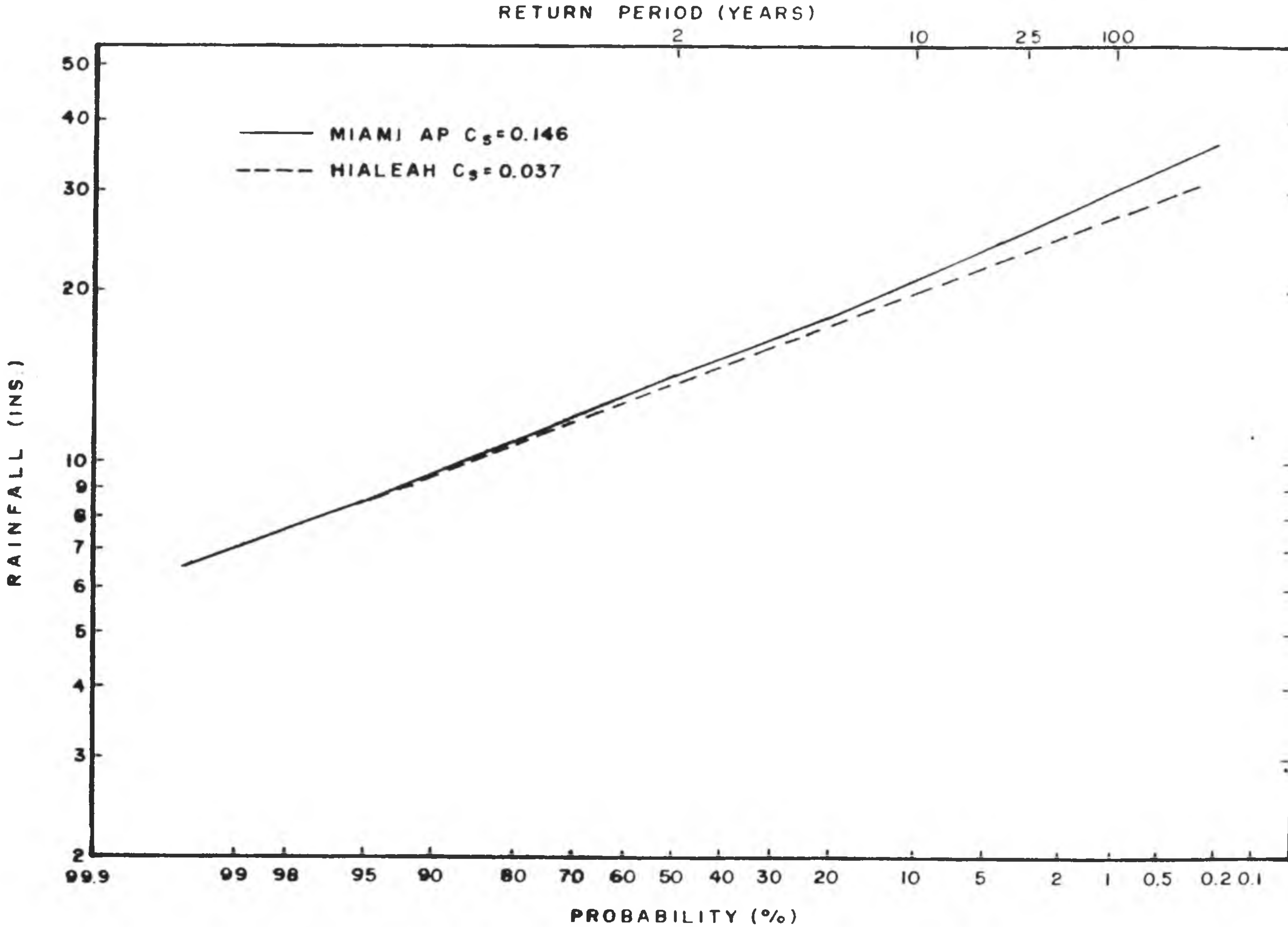
FOUR-DAY MAXIMUM PRECIPITATION

FIGURE 14



FIVE - DAY MAXIMUM PRECIPITATION

FIGURE 15



ONE-MONTH MAXIMUM PRECIPITATION

10-YEAR EASTERN DISCHARGE (CFS)

Storm (in) P	Q	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
		DAYS														
0.33	0.07	14	24	15	10	8	5	4	3	2	1	1	0			
0.60	0.24		48	82	51	34	27	18	13	11	7	5	3	1		
7.26	6.66			1332	2284	1405	939	739	493	366	293	206	133	73	27	
1.96	1.44				288	494	304	203	160	107	79	63	45	29	16	6
0.49	0.16					32	55	34	23	18	12	9	7	5	3	2
total		14	72	1429	2633	1973	1330	998	692	504	392	284	188	108	46	8
base flow		250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
total		264	322	1679	2883	2223	1580	1248	942	754	642	534	438	358	296	258
pumpage		0	111	111	111	111	111	111	111	111	111	111	111	111	111	0
total		264	433	1790	2994	2334	1691	1359	1053	865	753	645	549	469	407	258

TABLE 4

25-YEAR EASTERN DISCHARGE (CFS)

Storm (in) P	Q	DAYS 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.24	0.03	6	10	6	4	3	2	2	1	1	1	0	0			
0.9	0.48		96	165	101	68	53	36	26	21	15	10	5	2		
9.37	8.77			1754	3008	1850	1237	973	649	482	386	272	175	96	35	
2.40	1.87				374	641	395	264	208	138	103	82	58	37	21	7
0.43	0.12					24	41	25	17	13	9	7	5	4	2	1
		6	106	1925	3487	2586	1728	1300	901	655	514	371	243	139	58	8
base flow		250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
total		256	356	2175	3737	2836	1978	1550	1151	905	764	621	493	389	308	258
pumpage		0	111	111	111	111	111	111	111	111	111	111	111	111	111	0
total		256	476	2286	3848	2947	2089	1661	1262	1016	875	732	604	500	419	258

TABLE 5

100-YEAR EASTERN DISCHARGE (CFS)

Storm (in) P	Q	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
		DAYS															
0.13	0	0	0	0	0	0	0										
1.51	1.02		204	350	215	144	113	75	56	45	32	20	11	4			
13.16	12.55			2510	4305	2648	1770	1393	929	690	552	389	251	138	50		
3.23	2.67				543	916	563	376	296	198	147	117	83	53	29	11	
0.17	0.01					2	3	2	1	1	1	1	0				
+ base flow		250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
total		250	454	3110	5313	3960	2699	2096	1526	1184	982	777	595	445	329	261	250
+ pumpage		0	111	111	-	-	111	111	111	111	111	111	111	111	111	0	0
total		250	565	3221	5313	3960	2810	2207	1637	1295	1093	888	706	556	440	261	250

TABLE 6

SPF EASTERN DISCHARGE (CFS)

Storm (in) P	Q	DAYS 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.16	0.01	2	3	2	1	1	1	0	0								
1.89	1.38		276	473	291	195	153	102	76	61	43	28	15	5			
16.45	15.83			3166	5430	3340	2232	1757	1171	871	696	491	317	174	63		
4.04	3.47				694	1190	732	489	385	257	191	153	108	69	38	14	
0.21	0.02					4	6	4	2	2	1	1	1	1	0	0	
+ base flow		250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
total		252	529	3891	6666	4980	3374	2602	1884	1441	1181	923	691	499	351	264	250
+ pumpage		0	111	111	-	-	111	111	111	111	111	111	111	111	111	0	0
total		252	640	4002	6666	4980	3485	2713	1995	1552	1292	1034	802	610	462	264	250

TABLE 7

APPENDIX B

Routing

With estimated daily discharges in the east the next step in the analysis is translation to a backwater profile. Major inflow points along the main channel were determined from the permit inventory, USGS quadrangle sheets and aerial photography. Corresponding tributary areas for these channels were planimetered and weighted over the sub-basin area. The total discharge for each day was then distributed according to the weights at each inflow point. With such a discharge distribution along the channel, backwater profile computations were run with a District computer program employing the standard step method.

It was seen that at peak eastern discharge rates during the 100-year and SPF events the backwater stage reaching 67th Avenue far exceeds any possible stage in the west, even with the total rainfall depth added to existing ground elevations. With this stage difference east to west, there must be some westward flow.

This backflow rate is estimated by successive trial and error backwater computations. A trial point is selected east of 67th Avenue about which inflows divide east and west. The procedure involves two separate backwater profiles. One comes upstream from S-29 and the other is computed upstream eastward from the western sub-basin ponded area. If the profile from the west is lower at the dividing point then the profile from S-29 the point is moved eastward and inflows are diverted accordingly. This is continued until the profiles meet at the trial point. In this manner the backflow rate is found as a function of stage in the west and inflow rates in the east. To this westward discharge is added the pumpage as it should follow the flow orientation at its juncture with the main channel.

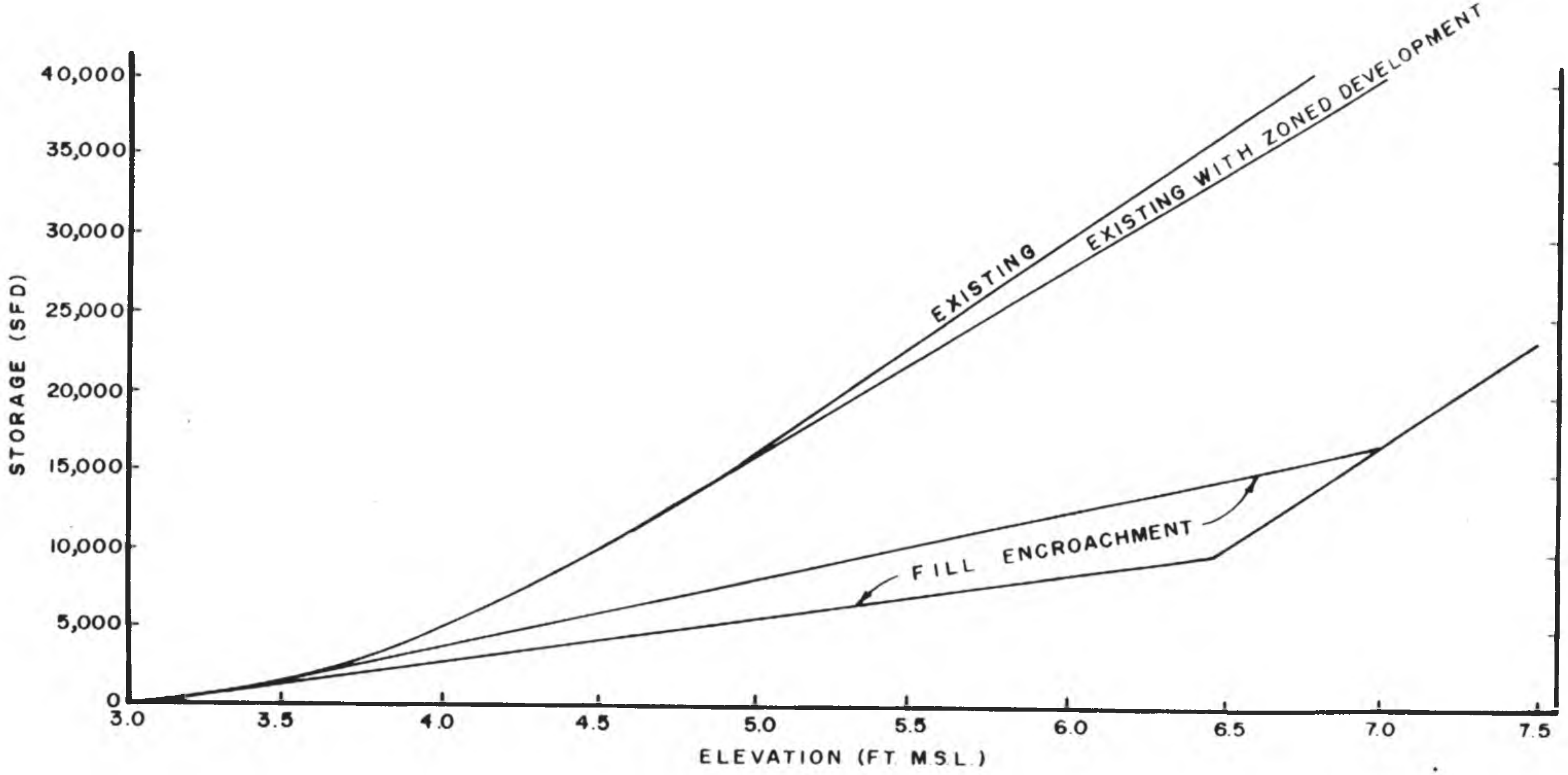
Because of the topography in the west and outflow dependence on eastern channel stages, simple reservoir routing is used in the western sub-basin. This is a primary hydraulic rather than a hydrologic approach since the physical relationships in the basin preclude any common hydrologic analysis.

Some important assumptions must be made for the western sub-basin routing. First, the antecedent groundwater stage is assumed at elevation 3.0 feet over the entire area. Examination of USGS published contours for the annual wet season show stages from about 2.0 feet to 5.0 feet. The highest stages are at the western end where the highest ground elevations are also found. 3.0 feet appears to be the average stage. With an assumed free storage coefficient of 0.20, a one-half foot different groundwater stage is not a significant volume of water. The second assumption is a perfectly flat water surface for free pool and groundwater across the basin. This is valid when the entire basin is inundated with more than one foot of water and little or no movement of water out of the basin. This assumption is less valid when water depths are small and discharge is occurring out of the basin Since the primary interest is in the peak stage which occurs under almost static conditions, the considerable complications of a more exact hydraulic model do not yield a commensurate amount of useful information.

In the western routings a daily budget period is used. Precipitation is converted to second-feet-day (SFD) volumes over the 27,600 acre area. Seepage is estimated as a constant 250 SFD. Evapotranspiration is estimated as 0.25 inch per day or 290 SFD. This is a rounded-off figure from nearby pan evaporation rates for August and September.

The stage-storage curve above 3.0 feet (Figure 16) is determined directly from Figure 3, the elevation distribution curve. The total storage for freely ponded water and groundwater was computed in 0.1 foot increments starting from 3.0 feet. Additional stage-storage curves are plotted for proposed development and fill conditions.

FIGURE 16



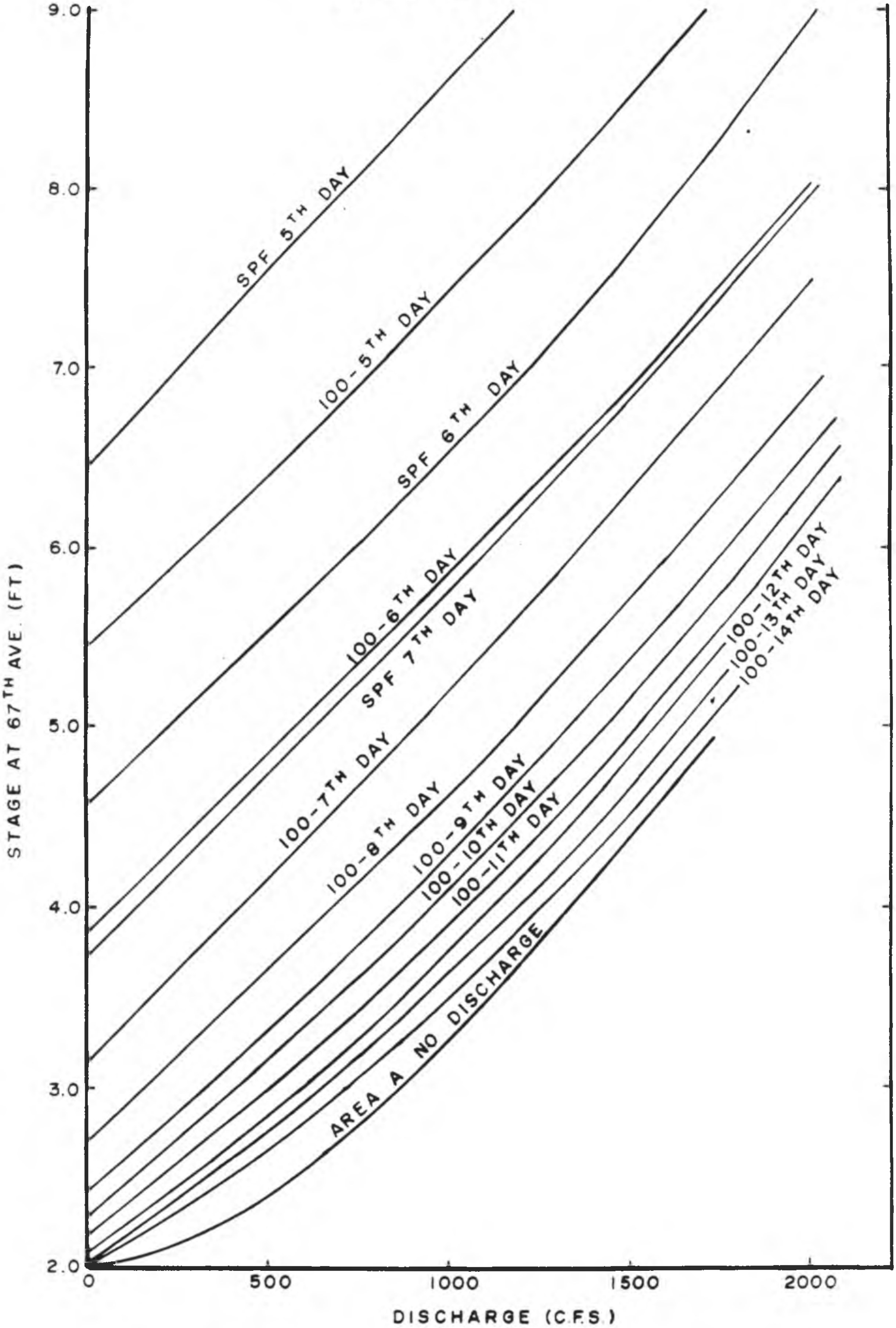
WESTERN SUBBASIN FLOOD STAGE-STORAGE

To each day's runoff condition in the east constant discharges from the west were superimposed and backwater computations performed. This yielded, for each day, a characteristic stage at 67th Avenue versus discharge out of the western sub-basin. This set of curves is shown in Figure 17. With these curves, a given ponded stage in the west will give a discharge from that area on each particular day.

The daily budget routings may now be performed. Input to storage is precipitation and seepage; out of storage is ET and discharge. The daily balance is net storage, and with the stage-storage curve an end-of-day stage is determined. The daily discharge rate is determined from the previous period end-of-day stage and the stage-discharge curves.

The routings for 100-year and SPF events are presented in Tables 8 and 9. Rainfall from the 6th to the 30th day is equal to the difference between the one-month and five-day frequency-depth determinations. For the distribution and proportion of these rainfalls the actual recorded rainfall at Hialeah on the days following the hurricane and flooding of 1947 is used.

FIGURE 17



DISCHARGE AT 67TH AVE.

100-YEAR EXISTING ROUTING

DAY	PRECIPITATION (in.)	PRECIPITATION (SFD)	SEEPAGE (SFD)	ET (SFD)	DISCHARGE AT 67th AVE (SFD)	STORAGE (SFD)	STAGE AT END OF DAY (Ft. MSL)
0	0	0	250	0	250	0	3.00
1	0.13	151	250	151	250	0	3.00
2	1.51	1754	250	290	250	1464	3.48
3	13.16	15287	250	290	250	16461	5.05
4	3.23	3752	250	290	-(1074+111)	21358	5.41
5	0.17	197	250	290	-111	21626	5.43
6	0.53	616	250	290	690	21512	5.43
7	0	0	250	290	1000	20472	5.34
8	0	0	250	290	1230	19202	5.25
9	0	0	250	290	1335	17827	5.15
10	0.17	197	250	290	1390	16594	5.05
11	0.33	383	250	290	1430	15507	4.96
12	0	0	250	290	1440	14029	4.86
13	0.52	616	250	290	1470	13133	4.78
14	0.03	35	250	290	1510	11618	4.65
15	0	0	250	290	1500	10078	4.53
16	0.03	35	250	290	1460	8613	4.39
17	0.43	499	250	290	1400	7672	4.31
18	0.67	778	250	290	1370	7040	4.25
19	0	0	250	290	1350	5650	4.12
20	0.03	35	250	290	1290	4355	3.98
21	2.13	2474	250	290	1230	5559	4.10
22	1.53	1777	250	290	1280	6016	4.15
23	0.78	906	250	290	1300	5582	4.09
24	0	0	250	290	1280	4262	3.97
25	0	0	250	290	1220	3002	3.80
26	0	0	250	290	1150	1812	3.57
27	0.62	720	250	290	1040	1452	3.50
28	2.06	2393	250	290	1010	2795	3.75
29	0.14	163	250	290	1125	1793	3.57
30	0.08	93	250	290	1040	806	3.28
31	0	0	250	290	900	134	3.00

TABLE 8

SPF EXISTING ROUTING

DAY	P (in)	(SFD)	SEEPAGE (SFD)	ET (SFD)	DISCHARGE AT 67th AVE (SFD)	STORAGE (SFD)	STAGE (SFD)
0	0	0	250	0	250	0	3.00
1	0.16	186	250	186	250	0	3.00
2	1.89	2195	250	290	250	1905	3.59
3	16.45	19109	250	290	250	20724	5.37
4	4.04	4693	250	290	-(1731+111)	27219	5.84
5	0.21	244	250	290	-(900 +111)	28434	5.93
6	0.66	767	250	290	600	28561	5.94
7	0	0	250	290	975	27546	5.86
8	0	0	250	290	1240	26266	5.77
9	0	0	250	290	1460	24766	5.66
10	0.21	244	250	290	1520	23450	5.56

TABLE 9

APPENDIX C

Encroachment Determination

The development of encroachment limits is based upon the condition that no backflow westward occurs during the peak runoff in the eastern sub-basin. As seen on the routing for the 100-year existing case the western sub-basin stage at the end of the third day is 5.05 feet. For a no backflow condition to occur this stage should be 7.02 feet, or coincide with the backwater profile which would be generated in the east if considered independently. Therefore, the encroachment fill volume should be equal to the difference in storage from 7.02 feet to 5.05 feet. This is taken from the existing stage-storage curve and is equal to 53708 acre-feet. Existing and planned development, 3520 acres, is considered to occupy 6730 acre-feet of storage below 7.02 feet and above existing ground elevations. The net allowable encroachment below 7.02 feet is then 46978 acre-feet. This is equivalent to 1.95 feet of depth over the area remaining with the existing and planned development. Round off to the nearest tenth of a foot would yield 2.0 feet of fill with 7.0 feet as the upper fill criteria level.

Placing this displacement volume into the lowest areas first will result in a perfectly flat basin at elevation 6.47 feet. This covers all existing ground. This type of filling would never be done since there would be no areas above a 100-year or even 10-year flood stage. The 100-year peak stage will not be changed no matter how the fill volume is placed as long as all of the allowed fill is below the 7.0 foot level. Of course, additional fill will be required above 7.0 feet for building pads but this will not enter the analysis, being above the water levels.

From the third to the fourth day in the 100-year routing, stages in the west peak at 7.32 feet because of additional rainfall with no discharge at the outlet. At this point, capacity has become available downstream and drainage of the western sub-basin begins. These peak stages and corresponding backwater curves to the east are given in Figure 4.

The second stage-storage curve developed is for the case of fill on the high lands first until the encroachment volume is exhausted. This would be the optimum arrangement from the landowners point of view as the maximum land area is brought up to 7.0 feet, all area above 3.9 feet, or about 64% of the total remaining basin.

The routings for the encroached 10-year, 25-year and 100-year events are given in Tables 10, 11 and 12. Only the 100-year event is carried through to the point when stages are back to antecedent levels. With the lesser events, the basic assumptions would make a total duration routing not very accurate.

10-YEAR STORM ROUTING WITH DEVELOPMENT

DAY	PRECIPITATION (in)	PRECIPITATION (SFD)	SEEPAGE (SFD)	EVAPOTRANS-PIRATION (SFD)	DISCHARGE AT 67th AVE (SFD)	STORAGE (SFD)	STAGE AT END OF DAY (ft.)
1	0.33	383	250	290	250	93	3.04
2	0.60	697	250	290	250	500	3.18
3	7.26	8433	250	290	250	8643	6.09
4	1.96	2277	250	290	1030	9850	6.49
5	0.49	569	250	290	1460	8919	6.20

TABLE 10

25-YEAR STORM ROUTING WITH DEVELOPMENT

DAY	PRECIPITATION (in.)	PRECIPITATION (SFD)	SEEPAGE (SFD)	EVAPOTRANS-PIRATION (SFD)	DISCHARGE AT 67th AVE (SFD)	STORAGE (SFD)	STAGE AT END OF DAY (ft.)
1	0.24	279	250	279	250	0	3.00
2	0.90	1045	250	290	250	755	3.27
3	9.37	10884	250	290	250	11349	6.60
4	2.40	2788	250	290	500	13597	6.77
5	0.43	499	250	290	1290	12766	6.72

TABLE 11

100-YEAR STORM WITH DEVELOPMENT

DAY	PRECIPIATION (in)	PRECIPIATION (SFD)	SEEPAGE (SFD)	ET (SFD)	Q (SFD)	STORAGE (SFD)	STAGE AT END OF DAY (ft MSL)
1	0.13	151	250	151	250	0	3.00
2	1.51	1754	250	290	250	1464	3.40
3	13.16	15287	250	290	250	16461	7.02
4	3.23	3752	250	290	-111	20284	7.32
5	0.17	197	250	290	850	19591	7.27
6	0.53	616	250	290	1550	18617	7.18
7	0	0	250	290	1760	16817	7.04
8	0	0	250	290	1960	14817	6.60
9	0	0	250	290	1910	12867	6.14
10	0.17	197	250	290	1800	11224	5.77
11	0.33	383	250	290	1730	9837	5.45
12	0	0	250	290	1660	8137	5.07
13	0.55	639	250	290	1530	7206	4.83
14	0.03	35	250	290	1480	5721	4.49
15	0	0	250	290	1390	4291	4.16
16	0.03	35	250	290	1230	3056	3.87
17	0.43	499	250	290	1180	2335	3.70
18	0.67	778	250	290	1090	1983	3.55
19	0	0	250	290	1030	913	3.31
20	0.03	35	250	290	920	0	3.00

TABLE 12

APPENDIX D

TABLE 13

WATER QUALITY ANALYSIS (mg/l)

U.S.G.S. STATION 43

Date	Total Dissolved Solids	Total P	Ortho-PO_4	Total N	NO_3	NH_4
9/30/70	-	0.02	0.01	0.52	0.1	0.33
1/14/71	397	0.02	0.00	0.56	0.1	0.15
4/7/71	388	0.01	0.01	1.0	0.2	0.28
7/20/71	394	0.01	0.01	2.1	0.0	0.08
9/22/71	-	0.01	0.01	1.2	0.1	0.26
1/6/72	364	0.02	0.00	1.8	0.1	0.32
4/4/72	390	0.02	0.01	1.2	0.2	0.23
6/15/72	-	0.03	0.00	1.2	0.1	0.12
9/27/72	386	0.01	0.01	1.4	1.9	0.21
1/16/73	385	0.00	0.00	0.94	0.1	0.21
3/20/73	392	-	0.01	0.89	0.2	0.19
6/19/73	394	0.00	0.00	0.91	0.1	0.14
8/29/73	-	0.01	0.01	1.7	0.0	0.25
9/24/73	-	0.03	0.03	1.0	0.2	0.25
1/18/74	-	0.02	0.02	1.4	0.1	0.16

Source: United States Geological Survey

TABLE 14

WATER QUALITY ANALYSIS (mg/l)

U.S.G.S. STATION 2862

Total Dissolved Solids	Date	Total P	Ortho-PO_4	Total N	NO_3	NO_2	NO_4	BOD
346	4/3/63	-	-	-	0.3	-	-	-
421	5/5/67	-	-	-	0.3	-	-	-
	1/6/72	0.02	.00	1.8	0.11	0.01	0.32	-
	1/11/72	-	-	-	-	-	-	1.2
	4/13/72	0.01	.00	1.2	0.10	0.00	0.12	-
	5/4/72	0.02	.01	1.2	0.20	0.01	0.23	0.5
	6/15/72	0.02	.00	1.2	0.10	0.00	0.12	0.7
	9/27/72	0.00	.00	1.4	1.9	0.00	0.21	1.0

Source: Water Resources Data for Florida, Part 2. Water Quality Records, United States Department of the Interior, Geological Survey, Page 415, 1972.

TABLE 15

WATER QUALITY ANALYSIS

Date of Collection	Mean Discharge (cfs)	Silica (SiO_2)	Iron (Fe)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Potassium (K)	Bicarbonate (HCO_3)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Dissolved Solids		Hardness as $CaCO_3$		Specific conductance in Mircomhos at 25^0C ($K \times 10^6$)	pH	Color
													Residue at 180^0C	Calculated	Calcium Magnesium	Non Carbonate			
							2.2861.8. SNAKE CREEK CANAL ABOVE S-30, NEAR HIALEAH												
5/8/67	277	4.4	.02	80	11	59	1.2	300	.0	91	.3	.4	441	396	246	0	760	7.7	50
10/4/63	0	7.0	.04	62	8.1	23	1.4	205	6.6	32	.7	.1	244		188	20	418	7.8	60
							2-2861.8E. SNAKE CREEK CANAL BELOW S-30, NEAR HIALEAH												
5/26/65	120	5.8	.00	94	6.2	53	.9	296	6.0	88	.5	.2		401	260	18	710	7.7	60
10/9/63	0	7.1	.05	70	6.7	25	1.4	232	4.8	34	.4	.5	280		202	12	462	7.8	70
							2-2862. SNAKE CREEK CANAL AT N. W. 67th AVENUE, NEAR HIALEAH												
5/5/67	112	5.2	.02	80	10	51	1.1	290	.0	87	.3	.3	421	379	242	4	738	7.8	50
4/3/63	81	4.8	.01	82	7.7	34	1.0	278	9.6	46	.4	.3	346	323	236	8	546	7.5	60

TABLE 16

WATER QUALITY ANALYSIS (mg/l)

BROWARD COUNTY ENVIRONMENTAL QUALITY CONTROL BOARD
STATION 7G

Date	PO_4	Total N	NO_3	NO_2 and NO_3	BOD
3/73	< 0.06	-	0.16	-	-
4/73	< 0.06	-	0.08	-	-
5/73	< 0.06	-	0.16	-	-
6/73	< 0.06	-	0.12	-	-
7/73	< 0.06	-	0.18	-	-
8/73	< 0.06	-	0.33	-	-
9/73	< 0.06	-	0.20	-	1.0
10/73	< 0.04	-	0.05	-	1.0
11/73	< 0.04	-	0.11	-	1.0
12/73	< 0.07	-	0.11	-	2.0
1/74	-	-	-	-	-
2/74	< 0.04	0.29	-	0.10	1.0
3/74	< 0.04	0.26	-	0.14	1.0
4/74	< 0.04	0.26	-	0.16	1.0
5/74	< 0.04	0.35	-	0.05	1.0
6/74	0.08	0.68	-	0.68	1.0
7/74	< 0.02	0.29	-	0.07	1.0
8/74	< 0.02	0.34	-	0.08	1.0
9/74	0.02	1.01	-	-	3.0
10/74	0.01	0.12	-	-	1.0
11/74	< 0.02	0.06	-	-	3.0
12/74	< 0.02	0.38	-	-	1.0

TABLE 17

WATER QUALITY ANALYSIS (mg/1)

BROWARD COUNTY ENVIRONMENTAL QUALITY CONTROL BOARD
STATION 7H

Date	PO_4	Total N	NO_3	NO_2 and NO_3	BOD
3/73	<0.06	-	0.76	-	-
4/73	<0.06	-	0.27	-	-
5/73	<0.06	-	0.14	-	-
6/73	<0.06	-	0.46	-	-
7/73	<0.06	-	0.33	-	-
8/73	<0.06	-	0.14	-	-
9/73	<0.06	-	0.55	-	1.0
10/73	<0.04	-	0.74	-	1.0
11/73	<0.04	-	0.73	-	1.0
12/73	<0.07	-	0.03	-	2.0
1/74	-	-	-	-	-
2/74	< 0.04	0.39	-	0.15	2.0
3/74	< 0.04	0.65	-	0.54	1.0
4/74	< 0.04	0.46	-	0.32	2.0
5/74	< 0.04	0.46	-	0.20	1.0
6/74	< 0.02	0.39	-	0.08	1.0
7/74	0.12	0.29	-	0.07	1.0
8/74	< 0.02	0.25	-	0.01	2.0
9/74	< 0.02	0.43	-	-	1.0
10/74	< 0.01	0.05	-	-	1.0
11/74	< 0.02	0.02	-	-	3.0
12/74	< 0.02	0.63	-	-	1.0

TABLE 18

WATER QUALITY ANALYSIS (mg/l)

BROWARD COUNTY ENVIRONMENTAL QUALITY CONTROL BOARD
STATION 7I

Date	PO_4	Total N	NO_3	NO_2 and NO_3	BOD
1/73	<0.06	-	<0.04		-
2/73	<0.06	-	<0.04		-
3/73	<0.06	-	<0.04		-
4/73	<0.06	-	<0.04		-
5/73	<0.06	-	<0.04		-
6/73	<0.06	-	<0.04		-
7/73	<0.06	-	<0.04		-
8/73	<0.06	-	<0.04		-
9/73	<0.06	-	<0.04		1.0
10/73	<0.04	-	0.03		1.0
11/73	<0.04	-	<0.04		2.0
12/73	<0.07	-	0.04		-
1/74	-	-	-	-	-
2/74	<0.04	0.23	-	0.03	2.0
3/74	<0.04	0.23	-	0.03	1.0
4/74	<0.04	0.50	-	0.29	2.0
5/74	<0.04	0.44	-	0.05	2.0
6/74	<0.02	0.54	-	0.22	3.0
7/74	<0.02	0.33	-	0.01	2.0
8/74	<0.02	0.74	-	0.44	7.0
9/74	<0.02	0.42	-	-	1.0
10/74	<0.01	0.01	-	-	1.0
11/74	<0.02	0.02	-	-	1.0
12/74	<0.02	0.44	-	-	4.0

TABLE 19

WATER QUALITY

STATION NO. L-1 (UPSTREAM)

Date	Time (A.M.)	Total Dissolved Solids cond/ppm	Total PO_4 as P/ppm	Total Nitrogen as N/ppm	Staff Reading MSL
9-4-74	10:00	310	0.04	0.3	-
9-11-74	10:25	320	0.04	1.1	3.67
9-18-74	10:35	320	0.02	0.7	2.84
9-25-74	10:55	310	0.02	0.8	4.31
10-2-74	11:05	320	0.02	1.4	3.70
10-9-74	10:25	320	0.02	1.0	3.93
10-16-74	11:15	330	0.06	0.7	3.46
10-23-74	11:05	330	0.06	0.4	2.72
11-6-74	9:55	355	0.02	0.3	2.04
11-21-74	10:30	340	0.02	1.0	2.62
12-4-74	10:00	324	0.02	0.4	3.67
12-18-74	10:15	325	0.06	1.1	2.75
1-3-75	10:05	340	0.08	0.5	2.51
1-16-75	10:20	275	0.02	0.3	2.94
1-29-75	10:10	295	0.02	0.8	2.52

Source: Paul R. McGinnes and Associates

TABLE 20

WATER QUALITY

STATION NO. L-1 (DOWNSTREAM)

Date	Time (A.M.)	Total Dissolved Solids cond/ppm	Total PO_4 as P/ppm	Total Nitrogen as N/ppm	Staff Reading MSL
9-4-74	10:20	335	0.02	0.3	-
9-11-74	10:40	295	0.02	1.4	2.88
9-18-74	10:50	270	0.02	0.8	2.72
9-25-74	11:05	295	0.01	1.1	2.89
10-2-74	11:15	295	0.01	1.5	2.67
10-9-74	10:35	300	0.02	0.6	3.00
10-16-74	11:25	295	0.03	0.5	2.67
10-23-74	11:20	320	0.01	0.4	2.56
11-6-74	10:10	285	0.02	0.6	2.58
11-21-74	10:40	350	0.01	0.8	2.65
12-4-74	10:10	330	0.01	0.4	2.73
12-18-74	10:25	300	0.05	0.6	2.53
1-3-75	10:20	350	0.03	0.6	2.38
1-16-75	10:30	350	0.00	0.2	2.42
1-29-75	10:20	360	0.01	1.0	2.52

Source: Paul R. McGinnes and Associates

TABLE 21

WATER QUALITY

STATION NO. C-9 (WEST)

Date	Time (A.M.)	Total Dissolved Solids cond/ppm	Total PO_4 as P/ppm	Total Nitrogen as N/ppm	Staff Reading MSL
9-4-74	10:30	345	0.01	0.7	-
9-11-74	10:45	340	0.01	1.2	2.85
9-18-74	10:55	340	0.00	1.0	2.61
9-25-74	11:10	330	0.00	0.8	2.85
10-2-74	11:20	335	0.01	2.2	2.67
10-9-74	10:40	340	0.01	0.8	2.98
10-16-74	11:35	345	0.01	0.6	2.68
10-23-74	11:25	350	0.01	0.4	2.58
11-6-74	10:15	345	0.01	0.4	2.59
11-21-74	10:45	350	0.01	0.9	2.64
12-4-74	10:20	350	0.01	0.5	3.03
12-18-74	10:30	350	0.03	0.8	2.94
1-3-75	10:25	365	0.03	0.3	2.84
1-16-75	10:35	365	0.01	0.4	2.88
1-29-75	10:25	360	0.01	1.1	2.90

Source: Paul R. McGinnes and Associates

TABLE 22

WATER QUALITY

STATION NO. L-2

Date	Time (A.M.)	Total Dissolved Solids cond/ppm	Total PO_4 as P/ppm	Total Nitrogen as N/ppm	Staff Reading MSL
9-4-74	10:40	325	0.01	0.5	-
9-11-74	11:00	295	0.02	1.6	2.86
9-18-74	11:05	310	0.00	0.8	2.63
9-25-74	11:15	310	0.00	1.6	2.79
10-2-74	11:30	320	0.02	1.6	2.61
10-9-74	10:50	315	0.02	1.0	2.95
10-16-74	11:45	315	0.02	0.9	2.65
10-23-74	11:35	310	0.01	0.6	2.58
11-6-74	10:20	345	0.05	0.8	2.60
11-21-74	10:50	350	0.01	1.0	2.60
12-4-74	10:30	320	0.01	1.0	2.27
12-18-74	10:40	310	0.04	0.9	2.11
1-3-75	10:35	345	0.02	0.6	2.06
1-16-75	10:50	350	0.00	1.1	2.47
1-29-75	10:45	355	0.02	1.1	2.53

Source: Paul R. McGinnes and Associates

TABLE 23

WATER QUALITY

STATION NO. C-9 (EAST)

Date	Time (A.M.)	Total Dissolved Solids cond/ppm	Total PO_4 as P/ppm	Total Nitrogen as N/ppm	Staff Reading MSL
9-4-74	10:50	345	0.01	0.4	-
9-11-74	11:05	330	0.01	1.8	2.94
9-18-74	11:10	340	0.00	0.6	2.70
9-25-74	11:20	330	0.01	1.0	2.84
10-2-74	11:35	340	0.01	1.8	2.66
10-9-74	10:55	335	0.01	1.8	3.00
10-16-74	11:50	340	0.01	0.8	2.72
10-23-74	11:40	345	0.01	0.4	2.67
11-6-74	10:30	345	0.01	0.8	2.68
11-21-74	10:55	350	0.01	1.0	2.70
12-4-74	10:35	345	0.01	1.6	3.08
12-18-74	10:45	350	0.03	1.2	2.93
1-3-75	10:45	355	0.02	0.6	2.80
1-16-75	10:45	355	0.00	0.9	2.47
1-29-75	10:35	360	0.01	0.8	2.52

Source: Paul R. McGinnes and Associates

TABLE 24

Wastewater Treatment Facilities
West C-9 Basin

	BROWARD COUNTY			DADE COUNTY	
	Heritage City Utilities	Hollywood Lakes Country Club	Haven Lakes Estates Mobile Homes	Country Club of Miami	Palm Spring North
sign Flow (MGD)	0.15	0.025	0.08	1.0	0.75
tual Flow (MGD)	-	0.020	0.088(9/75)	0.45	0.45
D5 - Discharge (mg/l)	-	3	17 (7/75) 44 (9/75)	3	10
- Discharge (mg/l)	-	4	8 (7/75) 105 (9/75)	6	10
al P - Discharge	-	2.8	>0.4 (7/75) 1.4 (9/75)	-	-
al N - Discharge (mg/l)	-	6.1	6.0 (7/75) 12.2 (9/75)	-	-
charges to	self contained lake	secondary canal	C-9	Spray Irrigation of Golf Course	Peter Pike Canal

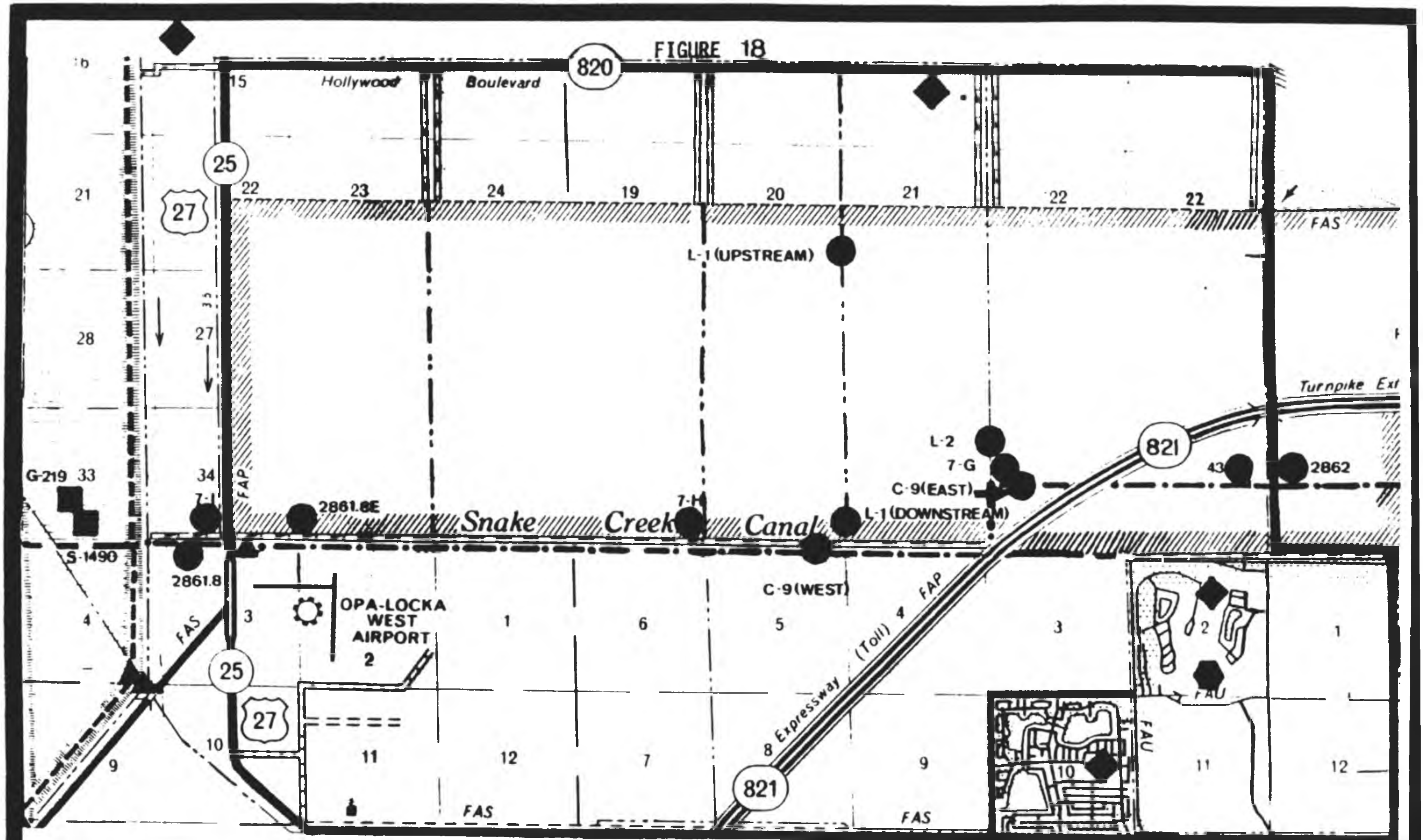
TABLE 25

Groundwater Quality Data
(mg/l unless otherwise noted)

Location	Date	Depth (ft)	Specific Conductivity (umho's)	Temp (^oF)	Silica (SiO_2)	Ca	Mg	Na	K	HCO_3	CO_3	SO_4	Cl	F	NO_3	Fe	TDS	Hardness Ca, Mg	Hardness Non Carbonate	Color	pH
G-219	9/18/41	32	398	83	-	61	10	5	-	214	-	1	19	-	-	-	-	193	-	110	-
G-219	9/19/41	56	426	77	-	70	8.3	8.2	-	242	-	1	19	-	-	-	-	209	-	110	-
G-219	9/23/41	90	877	77	-	50	28	91	-	321	-	25	105	-	-	-	-	240	-	20	-
G-219	9/24/41	134	1430	76	-	48	48	202	-	444	-	26	230	-	-	-	-	276	-	20	-
G-219	9/25/41	173	1640	76	-	42	42	257	-	458	-	33	282	-	-	-	-	249	-	20	-
G-219	9/26/41	198	2130	77	-	33	33	378	-	518	-	39	408	-	-	-	-	218	-	20	-
S-1490	4/20/64	45	660	72	5.8	94	8.1	40	0.8	304	0	5.2	66	0.3	0	1.3	371	268	19	60	7.3
S-1495	4/6/62	121	564	70	11	102	9.8	13	0.7	314	0	20	20	0	1.9	1.6	360	295	38	50	7.7
Country Club of Miami	2/7/74	-	-	-	-	-	-	-	-	-	-	16.9	20	0.3	<.01	1.1	446	-	-	79	7.3

	Turbidity 4JTU	Threshold Odor (units)	ABS	As	Ba	Cd	Cr^{+6}	Cu	CN	Mn	Phenols	Se	Ag	Zn
Country Club of Miami	4	2.8	0.25	<.005	<.01	<.005	<.01	<.01	<.01	0.019	<.01	<.01	.005	0.03

FIGURE 18



WATER QUALITY SAMPLING SITES

---- Canals and Drainage Ditches

Surface Water

Ground Water

Water Treatment Plant

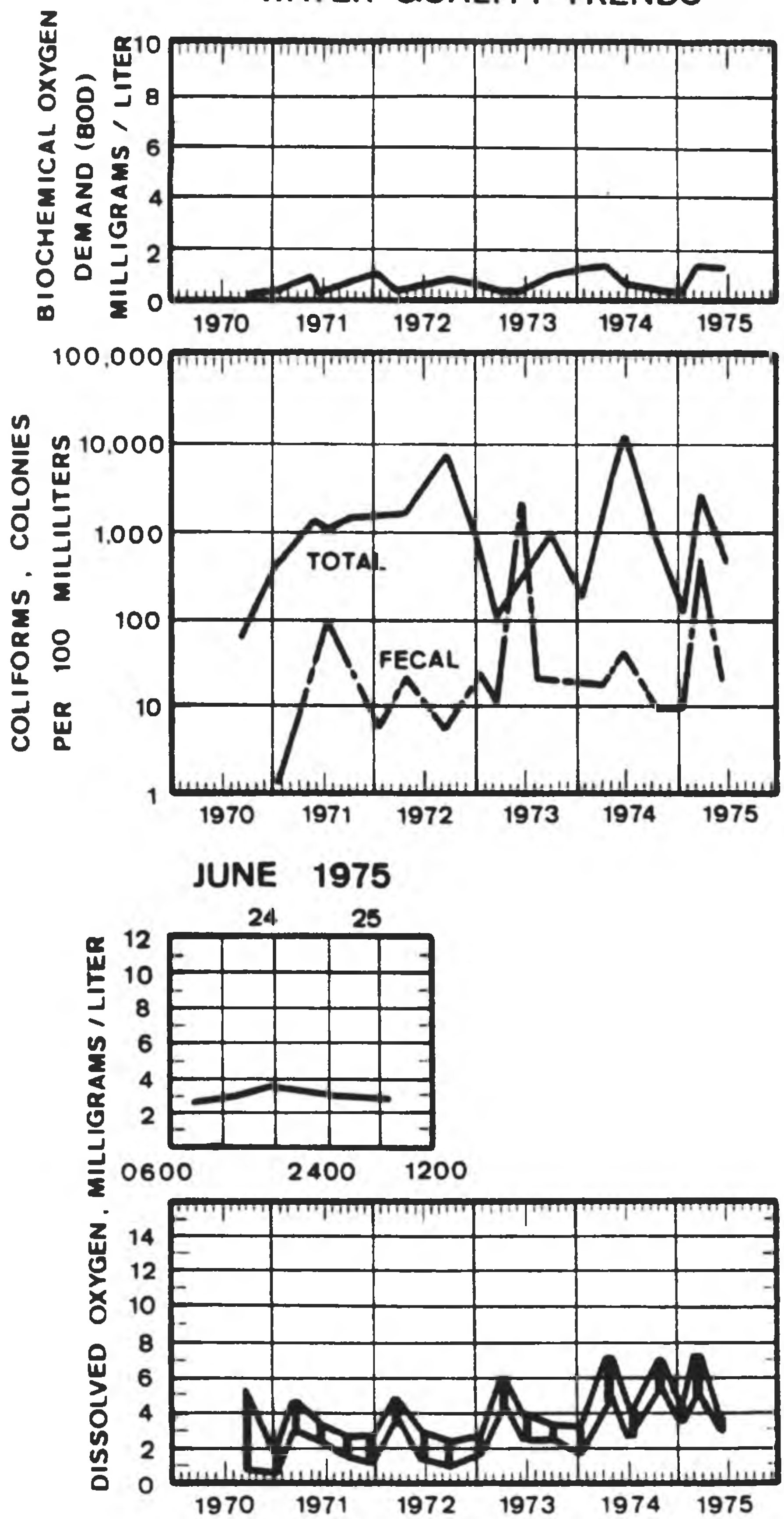
Waste Water Treatment Plant

Scale: 1" = 1 mile

N

FIGURE 19

WATER QUALITY TRENDS



Source: U.S.G.S. STATION 43

REFERENCES

1. Corps of Engineers, Part V, Supplement 12, Design Memorandum, Hydrology and Hydraulic Design, Canals in Greater Miami Area (C-2 Through C-9) dated March 23, 1954.

2. Corps of Engineers, Part V, Supplement 15, Design Memorandum, Snake Creek Canal Extension (C-9, Section 4), dated December 15, 1954.

3. Corps of Engineers, Part V, Supplement 27, General and Detail Design Memorandum, Canal 9 (Section 5) and Control Structure 30, dated April 20, 1959

4. Corps of Engineers, Survey-Review Report on Central and Southern Florida Project, Greater Miami Area (Area B), dated February 15, 1957, July 9, 1958 and August 24, 1961.

5. Soil Conservation Service, Technical Release No. 55, "Urban Hydrology for Small Watersheds", January 1975.

6. U.S. Geological Survey, "Analog Model Simulations for Secondary Canal Controls and Forward Pumping Water Management Schemes in Southeast Florida" 1975 (unpublished).

7. Weather Bureau, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, May 1961.

South Florida Regional Planning Council

1515 N.W. 167th Street, Suite 429, Miami, Florida 33169 (305) 621-5871

DATE 7-7-76
DIRECTOR
LAND ENG.
WATER MGMT.
ENV. SCI.
OTHER:
FILE:

- 7 1976
CENTRAL & SOUTHERN FLORIDA FLOOD CONTROL DISTRICT

July 6, 1976

J. Steve Reel, Director
Land Planning Division
Resource Planning Department
Central & Southern Florida Flood Control District
P.O. Box V
West Palm Beach, Florida 33402

Dear Steve:

In response to your letter of June 22, 1976, our staff has reviewed the draft of the Water Management Plan for the Western C-9 Basin. There are several questions regarding the draft, outlined as follows:

1. Although the C-9 Basin is essentially undeveloped at the present time, there are a variety of activities currently taking place within the study area, including farming. As the basin is developed under the proposed criteria, what will be the effect on these existing land uses?

2. Agriculture and related activities are permissible land uses under the proposed criteria; however, allowable encroachment for urban development may induce flooding in the non-urban activities. What provisions are to be made for insuring that one developer's encroachment is not another's headache?

3. Encroachment criteria do not specifiy the percentage of impervious surface which may be created by development, nor does it address water quality problems associated with such surfaces in the C-9 basin. Will this be covered under 208?

4. Water retention guidelines as outlined in the draft will apparently limit development within the C-9 basin very little. What impact will this have on the maintenance of the basin as an aquifer recharge area and as a stormwater retention basin for the eastern C-9 basin? Could development in the eastern C-9 basin potentially alter your position?

5. It was previously established that a concept of encroachment limited to available onsite fill would be more environmentally beneficial than one which allowed encroachment by means of imported fill materials. What is the rationale for the apparent change in this concept to the one presented in the draft report?

APPENDIX "E"

J. Steve Reel
July 6, 1976
Page 2

We appreciate this opportunity to review and comment on the draft report. These issues should be addressed in order to fashion the appropriate land management guidelines for the C-9 basin and surrounding Broward-Dade Sub-regional study area.

Sincerely,

Barry

M. Barry Peterson, AIP
Executive Director

MBP/as

Board of
COUNTY COMMISSIONERS
Broward County
FORT LAUDERDALE, FLORIDA

July 9, 1976

Transportation & Planning Departm
WATER MANAGEMENT DIVISIO
Broward County Courthouse
Room 530, 201 S.E. 6th Street
Fort Lauderdale, Florida 33301

Central and Southern Florida
Flood Control District
Post Office Box V
West Palm Beach, Florida 33402

ATTENTION Mr. J. Steve Reel, Director, Land Planning Division

Gentlemen:

RE: 9-1-2 - Water Management Plan for the Western C-9 Basin

Due to the limitation of time in which you wish us to comment on the above referenced report, we are limiting our comments to your final recommendations and not as to the assumptions and method of approach that you used to arrive at these recommendations. We will indicate our comments as follows in reference to page number of your draft:

Page 30 - Broward County's fill criteria does not dictate a floor pad elevation of at least 8.0 feet. Our current requirement is 9.0 feet which is our calculated 100-year flood stage. In any event, our current requirement is 18 inches above the crown of the nearest road or the 100-year flood level, whichever is higher.

Page 43: General Requirements (a) - As stated in the opening paragraph, we have not had sufficient time to check your calculations and assumptions, however, we would advise you at this time that our 100-year and 10-year flood frequency differs somewhat from what you have indicated.

Page 43: Design Requirements (a) - Based on current design practices and the cost differential involved, we believe that a three year design intensity should be sufficient for most road drainage systems.

Page 43: Design Requirements (b) - We are not quite clear as to *your intention as to where the first one-inch of runoff is to be retained. Is the first inch to be retained within some portion of a development's on site drainage system? If so, would lakes, canals, culvert systems or low lying areas be considered as allowable areas? Will secondary canals that discharge to C-9 be considered as part of the retention area?*

Page 44: Design Requirements (c) - We are not quite sure as to what your intent is in regard to the relationship between the 0.75 inch per hour pump discharge and the retention requirements of sub-paragraph (b), and although these matters might be considered in the actual design of the system, we see no way that they could realistically be enforced under actual field conditions because as soon as it is apparent that there is a potential flooding problem, most areas are going to begin to discharge and certainly it would be a few days after the storm before enough information is in to determine what the actual storm frequency was.

Page 44: Design Requirements (d) - If a development's drainage system is entirely isolated from others prior to discharging into a secondary canal, this office would probably require a higher elevation than 2.5 feet as a cut off point for discharging. This remark is based on our current fill criteria and could change after we completely review your calculations on the lower 10 and 100 year flood stages.

Page 44: Design Requirements (f) - At this time, we see no reason to change our minimum pipe size from 12 inches diameter, and of course, you could ask why we don't use a 10-inch minimum, and we could ask you why you don't use an 18-inch minimum.

Page 44: Design Requirements (i) and (j) - In regard to (i) and (j) and elsewhere in your draft, it has been our experience with several of the large developments that we have in similar areas within the adjacent C-11 basin, that the fill they obtain from on site for raising the land has almost made percolation a minor point in the consideration of the handling of storm water and due to the high water table and low fill criteria that you are proposing, it even becomes less of a dependable factor.

Page 45 (k) - The point that we will be bringing up here is similar to the diameter of the minimum size of pipe, and that is the slope of 7 to 1 that you propose. We have found that there is no difficulty whatsoever in walking on a 4 to 1 slope and changing from a 7 to 1 slope to a 4 to 1 slope is a 40% savings on land that would be required to be dedicated as a body of water or right of way and we would inquire as to what you are referring to when you speak of other excavations.

Page 48: Sub-Paragraph 3 - We have previously spelled out to you our current requirements for floor pad elevations, but we do not believe that the method of raising floors to the minimum elevation should be specified nor should it be limited only to the use of fill. We do not understand requirements for house pads being 6 inches above the regulatory flood level and in this respect we are assuming the regulatory flood level means the 100-year flood stage, and the continuation of the comment, "or the same as or higher than the street elevations." is totally unacceptable.

Insofar as the western area of C-9 is concerned in Broward County, we feel that consideration should be given in the fill requirements so that once the area is fully developed at some future date, that then the possibility of a second control in C-9 regulating the discharge eastward and maintaining a higher water level could be considered which would have a considerably beneficial effect on future water storage and aquifer recharge from the conservation area. Perhaps there is some development within the western limits of C-9 in Dade County that would preclude this from ever becoming a possibility, but we offer you this comment for your consideration.

Very truly yours,

J Stanley Weedon

J Stanley Weedon, P.E.
Director
Water Management Division

JSW/bp

909 S.E. First Avenue
Brickell Plaza, Building - Rm. 402
Miami, Florida 33131
Telephone: 579-2760

ENVIRONMENTAL RESOURCES MANAGEMENT

Water Control and Coastal Engineering Division

July 14, 1976

RECEIVED

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CENTRAL & SOUTHERN FLORIDA
FLOOD CONTROL DISTRICT

Mr. J. Steve Reel, Director
Land Planning Division
Resource Planning Department
Central and Southern Florida
Flood Control District
P. O. Box V
West Palm Beach, FL 33402

Dear Mr. Reel:

We have reviewed the Draft of the Water Management Plan for the Western C-9 Basin transmitted by your letter of June 22, 1976, and were favorably impressed with the thoroughness of the investigations and the professional manner in which the conclusions and recommendations were made. Our review was mainly confined to the technical aspects.

We do have some difficulty with Design Requirements, b., at the bottom of page 43. Rather than go into detail in this letter, we ask that you re-read the paragraph, and if it is clear to you maybe you could give us a call and explain it. Perhaps some minor re-wording is called for.

Very truly yours,

Charles C. Modisette

Charles C. Modisette
Acting Water Control Engineer

CCM:dfj
c.c.: Robert Usherson

JOHN R. MALOY, Executive Directo

CENTRAL AND SOUTHERN FLORIDA FLOOD CONTROL DISTRICT

P. O. BOX V
WEST PALM BEACH
FLORIDA 33402
Telephone (305) 686-S

IN REPLY REFER TO: 9-1-2

July 30, 1976

Mr. Barry Peterson, AIP
Executive Director
South Florida Regional Planning Council
1515 N.W. 167th Street, Suite 429
Miami, Florida 33169

Dear Barry:

Thank you for your considered comments of July 6, regarding the District's draft C-9 report. Below you will find individual responses to each of your questions.

1. Under the proposed criteria, as urban development increases in the western sub-basin, flood stages in that area may increase slightly. However, there should be a concurrent reduction in duration of inundation, which is a more important consideration in regard to agricultural activities.

2. Any land use activity could incorporate the allowable encroachment through the use of fill and/or dikes to prevent an increase in floodwater depth on the subject land parcel. The proposed criteria ensures that each parcel of developed land has the same relative volume of floodwater in storage after development as was in storage on the parcel in the existing undeveloped state.

3. The amount of impervious cover is not a significant factor in the water quantity analysis, since ground storage is almost insignificant under existing conditions. In terms of water quality, the proposed criteria include provisions for management practices which, when incorporated into drainage system designs, should result in improvement in storm water runoff quality. It is anticipated that these "Best Management Practices" will receive further attention as part of the up-coming Dade and Broward 208 programs.

4. Average annual discharge from the western C-9 basin is approximately 60 inches, and approaches 100 inches for some of the years examined. By comparison, the typical South Florida basin would exhibit a typical discharge of about 15 inches annually from 60 inches of rainfall. Therefore, the western C-9 basin behaves largely as a groundwater discharge area, resulting from high seepage rates from Conservation Area 3B. Local recharge from direct rainfall over the basin is, therefore, quite small when compared to recharge of the area from the conservation area system. The proposed criteria pertaining to surface water connections should, if implemented, reduce this groundwater drainage.

   In terms of storm water retention, the backflow of storm water from the eastern sub-basin to the west during the 100 year event and under existing development conditions is not necessary to prevent damages in the east. The encroachment prevents this backflow, but does not force discharge through the eastern portion of C-9 until there is adequate channel capacity available.

   With respect to further development in the eastern sub-basin, if secondary drainage systems in the eastern sub-basin are constructed and operated in accordance with existing regulatory agency criteria and normal practices with no significant drainage alterations installed in the areas already developed, there should be no reduction in flood protection in the eastern sub-basin.

5. The previous concept presented to members of the SFRPC at an early stage in our analysis, on January 27, 1976, was not one of encroachment but rather a limited cut and fill which would not modify the existing stage-storage relation at peak flood stage (Standard Project Flood). This early approach, one of several considered, was extremely conservative in setting the criterion that no increase in flood stage would result from existing to developed case. It was determined subsequently that a reasonable increase in flood stage in the western basin to a limiting point is possible without increasing flood damage in the east. This increase is brought about by encroachment fill obtained either out of the basin or within the basin. The encroachment effect is the same when on-site fill is obtained from below the groundwater stage.

If you have any questions regarding these issues, please do not hesitate to contact this office. Your letter and the District's response will be included in the final C-9 report.

Very truly yours,

Richard Rogers for

W. V. STORCH, P.E., Director
Resource Planning Department

WVS/srd

JOHN R. MALOY, Executive Dire

CENTRAL AND SOUTHERN FLORIDA

FLOOD CONTROL DISTRICT

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FLORIDA 3340
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IN REPLY REFER TO: 9-1-2

July 30, 1976

Mr. J. Stanley Weedon, P.E.
Director, Water Management Division
Transportation and Planning Department
Broward County Courthouse, Room 534
Fort Lauderdale, Florida 33301

Dear Stan:

Reference is made to your letter of commentary on the District's draft C-9 report. Following are our responses to each of your questions.

Page 30: The fill criteria for Miramar, and Dade and Broward Counties is at least eight feet. Broward County's criterion of 9.0 feet or 18 inches above the crown of the nearest road is certainly acceptable.

Page 43: General Requirements (a) - Although we acknowledge that your 100-year and 10-year flood elevations differ from ours, it must be recognized that our calculations and assumptions were based on information of considerable current site specific detail, particularly in regard to rainfall distribution and topographic information, as evidenced in pages 8-14 of the report.

Page 43: Design Requirements (a) - We agree that a three-year intensity design could be appropriate, and the report has been changed to reflect this.

Page 43: Design Requirements (b) - The intention is to retain the first one inch of runoff within some portion of the development's drainage system, and may include, for example, lakes, canals, culvert systems, low lying areas, and secondary canals behind control structures which serve the subject parcels.

Mr. J. Stanley Weedon
July 30, 1976
Page Two

Page 44: Design Requirements (c) - The only guiding factor in this criterion is the pump tailwater control, since in practical terms, the rainfall frequency and amount are not relevant if there is no rise in stage, such as might be the case for a non-basin wide event.

Page 44: Design Requirements (d) - We are not clear on the point you are making here. After your detailed review of our calculations for the 10-year and 100-year flood stages, please contact us to discuss this item.

Page 44: Design Requirements (f) - The minimum pipe size recommendation was based on FDOT type criteria. The wording in the report has been changed from "shall" to "should" to leave the option for local government to select minimum pipe sizes based on maintenance and other considerations.

Page 44: Design Requirements (i) and (j) - Percolation was considered as a minor factor in our analysis because of antecedent groundwater conditions.

Page 45: Design Requirements (k) - The language presented indicates that a 7:1 slope for water bodies should be required. It must be recognized that this is a current requirement in Dade County for rockpits and lakes, designed to accomplish two objectives: (1) public safety, and (2) improvement of storm water runoff quality. We concur with Dade County in striving to achieve these two objectives and encourage the 7:1 slope as a general guideline throughout the District. Certainly, however, there are circumstances when this could not be rigidly adhered to.

Page 48: Sub-Paragraph 3 - Regarding your first statement here, the report has been revised to indicate that other alternatives, in addition to fill, are acceptable in order to meet flood criteria. Concerning your second statement, the phrase "or the same as or higher than the street elevations" has been deleted. In addition, the requirement for house pads being 6 inches above the regulatory (100-year) flood level is a recognized safety factor which numerous communities and professionals recognize. The option to use such a criterion is a local decision and does not affect the District's analysis and recommendations nor should it discourage "stilt" construction which does not utilize building pads.

Finally, in regard to the possibility of installing a secondary control structure at some future date to regulate discharges to the east, we will keep such an option open for consideration at some time in the future, depending upon the degree and rapidity of development that occurs within the western C-9 basin.

We trust that the above satisfactorily addresses your comments and should you have any further comments or any additional questions on the C-9 study, please do not hesitate to contact this office.

Very truly yours,

Richard Rogers for

W. V. STORCH, P.E., Director
Resource Planning Department

WVS/srd

JOHN R. MALOY, Executive Di

CENTRAL AND SOUTHERN FLORIDA

FLOOD CONTROL DISTRICT

P. O. BOX
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IN REPLY REFER TO: 9-1-2

July 30, 1976

Mr. Charles C. Modisette
Acting Water Control Engineer
Water Control & Coastal Engineering Division
Dept. of Environmental Regulation Management
909 S.E. First Avenue
Brickell Plaza
Miami, Florida 33131

Dear Charles:

Thank you for your complimentary letter regarding the District's draft C-9 report. In regard to the comment concerning Design Requirements (b) on page 43, Mr. Richard Rogers will contact you, if he hasn't already, to clarify the paragraph.

Essentially, a typical example would be:

a. The maximum one hour rainfall from a 3-year 24-hour storm using the appropriate Florida Department of Transportation rainfall intensity duration curves and a runoff coefficient of 0.4 would require 1.04 inches of runoff storage.

b. The runoff storage required for five hour average detention for a 25-year 24-hour storm, using the same rainfall and runoff coefficient would be 0.86 inches.

These compare favorably with the one-inch retention number for most projects of moderate development intensity. As the intensity increased, if the result was an increase in the runoff coefficient, the storage requirement would increase accordingly, which seems logical. Our experience has shown that storage requirements for quantity management purposes usually exceed those herein discussed for quality management purposes.

If you should have any additional questions, please contact this office.

Very truly yours,

Richard Rogers for

W. V. STORCH, P.E., Director
Resource Planning Department

WVS/srd