EVALUATION OF THE WEST PALM BEACH CANAL WATERSHED TRIBUTARY TO PUMPING STATION S-5A

by

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ABSTRACT

The hydrologic regime of the 221 square mile watershed tributary to Pumping Station 5A on the West Palm Beach Canal was examined. Rainfall records from stations within and adjacent to the basin were used to determine one-day and two-day rainfall frequencies. These frequencies were assigned to twenty selected rainfall events having fairly uniform distribution over the basin.

Rainfall-runoff relationships were determined for each of the storm events, as were average monthly and average annual relationships for the period of record. The derived monthly rainfall loss values were verified against data developed by the Agricultural Research Service.

Values of Manning's roughness coefficient were computed from a number of discharge measurements and hydrograph analyses. Channel bottom profiles and cross-sections were taken. These, together with the values determined for Manning's "n" were used to analyze the performance of the system during two major storm events.

For the storms of March 25, 26, 1970 and June 29, 30, 1966, inflow and discharge hydrographs at selected locations were developed. The 1970 event had a recurrence frequency of once-in-ten years, and the 1966 event a frequency of about once-in-five years; the design condition. The analysis of the performance of the S-5A primary system included consideration of the extent of agricultural development, status of local secondary drainage works, and operation of the S-5A pumping facilities.

The system was found to have performed satisfactorily during the 1966 event and also during the more severe event (in excess of the design condition) of 1970. It is concluded, however, that the system is now at about

the maximum limit of its capabilities with about 25% of the watershed still not under secondary pumped water control.

Measures which could be taken to maintain satisfactory performance under conditions of maximum watershed development and perhaps improve performance, were examined. The measures considered were channel cleanout, modified pumping operations procedures, application of a revised formula for allocating secondary inflow, and action to reduce installed pumping capacities in a specific area tributary to Cross Canal.

In regard to channel cleanout, water surface profiles were recomputed based on actual channel dimensions and roughness coefficient. It is estimated that the design profile could be lowered by about one foot in the upper end of the basin by restoring the design channel section.

Pumping procedures during the 1966 and 1970 events were examined. It is concluded that performance can be improved by accomplishing an early drawdown of intake stage at S-5A and that such drawdown can be obtained even under conditions of excessive rainfall.

Existing secondary farm pump installations were located and sizes determined. These were analyzed in terms of type of agricultural land use. These data, together with data resulting from analysis of the two storm events, were used to derive a new curve for local inflow allocation.

Pumping installations now in existence in the Gladeview Canal area were compared with those serving other portions of the watershed. These installations are found to be excessive in capacity and that the installed capacity is not justifiable.

Recommendations are made to undertake channel cleanout under an orderly, phased program; to modify pumping operations procedures and criteria; to apply and implement the revised local inflow allocation formula; and to take action to reduce the size of the existing pumping installations tributary to Gladeview Canal.

Recommendations are also made to implement similar programs, as necessary, in the other watersheds of the Everglades Agricultural Area.

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The report was written by W. V. Storch, with review by S. Lin, R. L. Taylor, and Z. C. Grant, P.E., Director of the Department of Field Services.

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INTRODUCTION

Scope and Purpose of Investigation

The basic elements of the primary water control system for the S-5A watershed have been in operation since 1956. During this period the system has been subjected to several tests of varying degrees of severity. Subjectively, it appeared that the system performed well under those circumstances since no problems associated with system and/or operational deficiencies were readily identified.

However, during this period only one study of system performance, as such, was made. This was an examination by the District of the December 26, 1957 event and was general in scope, and thus limited. It was a simple comparison of the actual time taken to remove from the basin the excess rainfall from this event with the time it would have taken had S-5A not been in existence.

In view of the lack of any detailed objective flood control performance analysis it was decided in 1972 to undertake an in-depth study of the performance of the S-5A primary water control facilities in the context of the S-5A watershed system. That system, in this broader sense, includes such elements as type of land use, extent of land development, character and type of secondary drainage, hydrologic regime, hydraulic characteristics of the primary facilities, institutional controls on runoff regulation, and operational practices.

The purpose of the study is to relate facility performance to facility design and to those other identifiable factors of the watershed system which can affect performance. Incorporated within this purpose is the determination, insofar as possible, of the nature of the relationship between performance and the factors affecting performance.

The over all objective of the study is to develop understanding and knowledge of an integrated agricultural watershed system under pumped primary water control. The S-5A watershed was selected because of its relatively long history of operation, its high degree of agricultural development, and its comparatively good hydrologic data base.

The location of the study area is shown on Plate 1.

Descriptive Background

The West Palm Beach Canal was constructed in the period 1913 to 1929, between Lake Okeechobee at Canal Point and Lake Worth at West Palm Beach. It was one of the four major canals of the Everglades Drainage District traversing the Everglades basin between Lake Okeechobee and the lower southeast coast.

Under the Central and Southern Florida Flood Control Project, Pumping Station 5A was constructed at Twenty Mile Bend, the point, twenty miles from the Lake, at which the West Palm Beach Canal turns east. This is also the point at which the general character of the surface soils change from organic mucks and peats on the west to sands and shells on the east.

Pumping Station 5A (S-5A) was completed and placed into operation in early 1955. It diverted surplus storm runoff from agricultural mucklands, which formerly discharged to tidewater, into Conservation Area No. 1. Prior to 1955 the average annual discharge of West Palm Beach Canal to tidewater approximated 771,000 A.F. Since 1955 the average annual discharge has been about 487,000 A. F.

Associated with S-5A at Twenty Mile Bend is a set of water control structures termed the "S-5A Complex." Designated S-5A(E), S-5A(W) and

S-5A(S), these controls permit a certain degree of flexibility to be exercised in manipulating flows converging at Twenty Mile Bend from the West Palm Beach Canal both east and west of S-5A, and from the Levee 8 canal entering from the north.

The L-8 canal is designed to discharge to Conservation Area No. 1 by gravity through S-5A(S). However, by closing S-5A(S) and opening S-5A(W), L-8 flows can be pumped at S-5A. This is done since the L-8 area cannot tolerate the high stages which are often required to obtain gravity flow to the conservation area. By similar manipulation flows in the West Palm Beach Canal east of S-5A can also be partially reversed and pumped at S-5A. This is done by opening both S-5A(E) and S-5A(W), while maintaining S-5A(S) closed. The S-5A complex is shown in the insert on Plate 2.

As a feature of the Central and Southern Florida Project, the West Palm Beach Canal between S-5A and the Lake was enlarged and leveed on both sides. Under the Project designations Levee 10 and Levee 12, this canal and levee work was completed in 1956.

The Cross Canal, located between the Hillsboro and West Palm Beach Canals, is a major contributor of inflow to West Palm Beach Canal. Because of its restricted section and its proximity to S-5A, erosive velocities were experienced at the U. S. 441 bridge crossing shortly after S-5A was placed in operation. In 1955 the District installed a rock weir downstream of the bridge to control velocities. Also to regulate flows to S-5A, a control structure, S-5A(X), was installed in Cross Canal in 1956 at the divide between the Hillsboro and West Palm Beach Canal basins.

The rock weir remained in place until 1969 when it was removed, the highway bridge was re-built, and the eastern 1.3 miles of Cross Canal (L-13) was enlarged by the Project.

Pertinent Physical Information

The area of the S-5A watershed is 221 square miles. Of this total area, 61 sq. mi., or 27%, is tributary to Cross Canal (L-13). The watershed is entirely overlain by organic soils between five and ten feet in depth. Ground elevations vary from 13.0 ft. msl. to 18.0 ft. msl.

As of 1970, approximately 90% of the watershed had been placed into some type of productive agricultural use. Sugar cane, truck crops and improved pasture are the major agricultural land uses in the watershed. About 48% of the watershed area is devoted to sugar cane, 16% to truck crops, and 20% to improved pasture. (See Table 1). Urban land use is negligible. The 1970 land use pattern is shown on Plate 2.

On-the-land flood control and drainage is accomplished in large part by means of farm pumps, which are privately owned and operated, discharging to the primary canal facilities. As of 1970 the total acreage under farm pumps approximated 160 sq. mi. Farm pumps and their locations are listed in Tables 2 and 3. Drainage from the remainder of the watershed is delivered by gravity discharge to the primary system. Approximately 30 sq. mi. of the watershed's developed lands are served by improved secondary gravity drainage systems. The remaining 21 sq. mi. of undeveloped lands drain naturally, by gravity, to the primary system. The 1970 secondary drainage system is shown on Plate 3.

The typical method of connecting secondary on-the-land flood control and drainage works to the primary canal system is by means of pipe culverts. Gravity inflow can be regulated by culvert size; pumped inflow by both culvert size and pumping capacity. In the S-5A watershed, as in the remainder of the Everglades Agricultural Area, allowable inflow (and hence culvert and/or pump capacity) is determined by application of the "Everglades Runoff Formula." In this watershed, however, many secondary drainage connections were made prior to the advent of the Flood Control District and its application of the above formula.

Cross Canal inflow to the West Palm Beach Canal is unregulated except by the dimensions of its channel. With one major exception, all secondary drainage inflow to both the West Palm Beach and Cross Canals is regulated to a greater or lesser degree by means of pipe culverts or similar structures. The one exception is the Gladeview Canal which enters Cross Canal about 4 miles above the latter's junction with the West Palm Beach Canal. Gladeview Canal passes through a box culvert, carrying S. R. 80 over the canal, immediately upstream of its confluence with Cross Canal. This structure provides a limited degree of flow regulation.

The primary system for the S-5A watershed was designed to remove 3/4 inch of runoff per day from the 221 sq. mi. tributary area. It was considered that this peak runoff removal rate would protect the service area from any damaging flooding consequent to a two-day rainfall having an average recurrence frequency of once every five years. Under such circumstances it was calculated that peak stage in the West Palm Beach Canal at Canal Point (its upstream end) would approximate 13.7 ft. msl. The design was predicated on full agricultural development of the watershed, including comparable improvement (at least 3/4 inch per day runoff removal) of secondary drainage systems.

Pumping Station 5A contains six pumps, each having a rated capacity of 802 cfs. With all pumps operating at rated capacity, the volume of water moved in a 24-hour period is equivalent to a 3/4 inch depth of water over the service area of the pumping station. Under design conditions a draw-down stage of 8.3 ft. msl. was established at the intake side of the pumps. Pump design was based on a discharge stage in Conservation Area No. 1 of 17.5 ft. msl. Design operating speed was 714 rpm. Pumping capacity is dependent upon pool-to-pool head and engine speed.

Structure S-5A(W) is a gated twin barrel 7 ft. x 7 ft. box culvert designed to pass a design discharge of 700 cfs. S-5a(S) is a spillway with two 19.33 ft. x 22.8 ft. vertical lift gates designed to pass L-8 canal flows by gravity into Conservation Area No. 1 when sufficient head is available. S-5A(E) is identical to S-5A(W). S-5A(X), the divide structure in Cross Canal, consists of 4 - 72" diameter culverts with gates.

Normal water control stage in the West Palm Beach and Cross Canals during the wet season is 12.0 to 12.5 ft. msl. The usual operating procedure with the advent of rainfall is to draw down the stage at the S-5A intake to between 9.5 and 10.0 ft. msl., placing additional pumps on line, as necessary, to maintain this stage. Stages at Canal Point being the most critical, these are used as an indicator to guide pumping rates at S-5A and the timing for introduction of water to the S-5A intake either from L-8 or from east of S-5A(E), if conditions warrant.

The normal practice is to cut back pumps or reduce engine speeds when peak stage at Canal Point firmly evidences recession. If at this point high stage conditions still prevail in either L-8 canal or in West Palm Beach Canal east of S-5A(E), then, in lieu of cutting back pumps or engine speed, available pumping capacity at S-5A is used to relieve flooding conditions in those areas.

Neither the L-8 canal area nor the area tributary to West Palm Beach Canal east of S-5A(E) is within the designed service area of S-5A. However, the system has sufficient flexibility to afford relief to those areas by means of S-5A if necessary. The general operating rule is to give priority of service to the S-5A watershed proper. Nevertheless, secondary relief is provided the other areas, when conditions warrant, even though rainfall excess originated on the S-5A watershed itself has not been completely removed.

EVALUATION OF HYDROLOGICAL AND HYDRAULIC DATA

Source of Data

The period of record used in this study was 1955 - 1970.

Rainfall information from 13 rain gages in and near the study area was used. The location of the pertinent rainfall stations is shown on Plate 3.

Canal stage information was obtained from continuous recorders in the West Palm Beach Canal located at Canal Point, Big Mound Canal and S-5A. Stage recorder locations are also shown on Plate 3.

Discharge information at S-5A and S-5A(W) was obtained, or derived, from District operational data and from "Water Resources Data for Florida" published by the U. S. Geological Survey. Although no discharge data at S-5A(X) is available, it was assumed there was no eastward flow at this point during the storm events analyzed.

Analysis of Rainfall Events

Twenty rainfall events whose distribution over the watershed was fairly uniform were examined. The majority of these events occurred during the dry months of November through May, with only six events falling in the normally wet months of June through October.

The frequency of occurrence of rainfall events over the watershed was analyzed by using the rainfall data obtained at five stations; Pelican No. 34, Pelican No. 23, Canal Point, Pahokee No. 2 and S-5A. These stations were selected because of the longer period of record available for analysis.

The frequencies for one-day and two-day maximum rainfall events were determined for each station from these data. The plots of rainfall depth vs. frequency of occurrence for one-day and two-day rainfalls are shown on Plates 4 and 5, respectively. The formula used for plotting positions is:

$$P = \frac{m - 0.3}{N + 0.4}$$

where: m = the order number of the event

N = the number of years of record used.

The plots indicate that the variations in both one-day and two-day rainfall depths over the entire basin for a once-in-five year frequency event approximates 1 inch. The apparent variation becomes greater for events less frequent than this.

For the twenty selected rainfall events examined, the Thiessen method was used to compute the average rainfall depths over the basin. A frequency was assigned to each event by use of the curves shown on Plates 4 and 5. The events examined, the rainfall depths for each event, and the assigned event frequency are listed in Table 4.

The June 29-30, 1966 event had a recurrent frequency of close to once-in-five years; an event on which the design of the S-5A system was based. The March 25-26, 1970 event had a recurrence frequency of once-in-ten years; it represents the severest test which the system has experienced to date.

The two-day rainfall, having a once-in-five year frequency, which was used by the Corps of Engineers for system design, was 7.3 inches. This is to be compared with 6.5 inches derived from the analysis herein.

Rainfall-Runoff Relationships

A generalized overall basin relationship between rainfall and runoff was developed for each of the twenty rainfall events examined. Basin runoff was computed by an inflow - outflow procedure. The computational procedure is illustrated by Table 5, which gives the runoff values for each day until the rainfall excess generated by the June 1966, and March 1970, events was completely removed from the basin. It is to be noted that for the approximate five-year storm event of June 1966, the peak days show a basin runoff removal rate of just at 3/4 inch per day; the design value. Basin runoff removal rate on the peak day of the ten-year storm event of March 1970 was just under 1 inch; indicating a system capability in excess of the design.

Total basin runoff, similarly computed for each event examined, is listed in Table 4 together with a runoff-rainfall ratio. As is to be expected, there is a wide variation in this ratio which is principally explainable by: (a) antecedent soil moisture conditions and (b) state of secondary drainage development.

The rainfall-runoff data from Table 4 are plotted on Figures A, B and C of Plate 6. Figure A is the data from all twenty events, Figure B the data for the dry month events, and Figure C the data for the wet month events. These plots indicate that during the dry months a one-inch rainfall will produce little or no runoff whereas about 0.4 inches of runoff will result during wet months. With rainfall amounts greater than 5 inches about the same amount of runoff will be generated regardless of when it occurs.

Rainfall Excess Evaluation

Rainfall excess is another term for runoff; or that portion of the precipitation from a single event which shows up as streamflow in excess of base flow. During any rainfall event a portion of the precipitation will go into surface detention, groundwater storage and increased soil moisture content. A large portion of these temporary storages will eventually be lost through evapotranspiration.

The monthly rainfall-runoff relations for the S-5A watershed were examined in order to provide some basis for estimating monthly rainfall excess, or monthly water loss. The period January 1961 through December 1969 was used and the results are shown on Plate 7. The computational procedure was the same as that used for the rainfall-runoff analysis of the individual storm events.

The Agricultural Research Service of the Soil Conservation Service, USDA, has prepared a rainfall-excess evaluation for the Everglades Agricultural Area (Appendix A, Part VI, General Studies & Reports, Section 8). The results of this analysis are shown on Plates 8 and 9. The data points for the S-5A watershed are shown together with those of the ARS. The S-5A data show somewhat more runoff for a given amount of rainfall for June, July and August, but about the same relationship for September and October. It is considered that the ARS relationships for monthly water loss are generally valid for the S-5A watershed.

Plate 10 is another representation of the monthly runoff for the period examined, developed in annual form. The annual values of rainfall and basin runoff is shown. These indicate that a high percentage of annual precipitation shows up as runoff pumped at S-5A. This is to be expected from a watershed such as this which is highly developed and in which a high percentage of the land is under positive agricultural water control. Plate 11 shows the annual relationship between rainfall and runoff.

The U. S. Geological Survey ("Hydrologic Effects of Water Control and Management of Southeastern Florida", 1972) has developed a general relationship for watersheds in southeastern Florida which indicates that an annual rainfall of 37 inches or less will produce no effective runoff. This does not appear to be applicable to the developed S-5A watershed, based on the data presented on Plates 10 and 11. A value of about 34 inches appears, tentatively, to be valid. If this value is assumed, then the data presented show that for this basin, it is quite probable that all annual rainfall above this amount will appear as runoff (discharge) at S-5A. In six of the nine years examined, this is the case. (See Plate 10).

The accumulated annual rainfall and runoff curves shown on Plate 10 indicate a fairly long-term stable hydrologic regime for the S-5A basin.

Pumping Characteristics

As stated earlier, the design capacity of each of the six pumps at S-5A is 802 cfs at a maximum static head (pool to pool) of 11.1 feet. This head at which maximum efficiency is obtained is 9.2 ft. Design engine speed is 714 rpm.

Based on manufacturer's data and pumping tests prior to acceptance of the pumping equipment in 1955, a set of pump rating curves was developed. A series of discharge measurements at S-5A was made by the U. S. Geological Survey starting in 1957. These discharge measurements were made by use of a deflection meter (flow velocity determinations). The velocity determinations were then related to stage and the cross-sectional area of the channel section to develop discharge values. The U. S. Geological Survey ratings were used for this study since it is believed these provide a better definition of discharge because of the extensive field data on which they are based. The rating curves are shown on Plate 12.

One of the determinants of pumping capacity is the static head under which the pumps operate. With a reduced head, a greater discharge than design will result. Tailwater (Conservation Area No. 1) stages at S-5A were examined for the period January 1962 to December 1969. Plate 13 is the stage-duration curve for the discharge side of S-5A. This shows that S-5A tailwater was at or above the design stage of 17.5 ft. msl., less than 1% of the time during the period examined. It can be expected, therefore, that during the great majority of pumping operations greater-than-design pumping capacity can be developed, if necessary, due to the probable lower pumping head.

This is borne out by the discharge-duration curve on Plate 14. Discharges for the 1958 through 1970 water years were used to develop this curve. This shows that S-5A has, on occasion, discharged at rates in excess of the design rate of 4800 cfs., (about 0.2% of the time). In fact, a maximum discharge rate of about 5250 cfs, 10% over design, has been developed on these occasions.

Plate 14 also compares the actual experience of pumping duration with that estimated for S-5A system design. The design, or predicted, discharge-duration curve indicates that the station would be operating at full capacity 10% of the time. Actual experience, based on the 13 year period examined, shows operation at full capacity, or above, only about 0.2% of the time. The comparison also shows more actual extremely low discharge rate pumping than predicted. The overall picture, however, is one of substantially less pumping than initially contemplated. Consequently, actual pumping cost, i.e., the annual cost of primary water control for the S-5A watershed, is less than originally estimated.

Discharge duration is also a measure of basin water yield. A 14-year period was used in the design phase as the basis for determining the design discharge-duration relationship. The average annual rainfall for this period was 57.6 inches based on the Belle Glade records. The period examined to develop the actual discharge-duration curve was 13 years; with an average annual rainfall of 55.2 inches. The two periods are therefore comparable in terms of rainfall amounts. The comparison of the two curves indicates that there has been more retention and conservation of water within the S-5A watershed than predicted on the basis of runoff records prior to construction of

S-5A. It can be reasonably assumed from this that the S-5A flood control system has acted also to conserve water within the watershed.

Pumping characteristics in terms of relating rainfall to maximum daily discharge were also examined. Plate 15 shows the result of this analysis in the form of a rainfall-maximum daily discharge curve. The rainfall depths for each of the twenty selected events were plotted against the maximum daily discharge (expressed as cfs per sq. mi.; CSM) measured at S-5A during the course of the event. The curve shows that for about 5.5 inches of rainfall a discharge of 20 CSM results. This rainfall amount is about 1.0 inches less than the five-year frequency rainfall derived herein (see Plate 5). The maximum daily discharge of 20 CSM is equivalent to 3/4 inch over the entire basin; the design runoff removal rate. This curve demonstrates that design discharge rates at S-5A can be expected to be achieved with somewhat less than the design storm rainfall over the basin.

This curve provides the fundamental relationship, for storm events, between rainfall depth and S-5A discharge which will prove useful in the future development of simulation models for pumped agricultural watersheds in the Everglades Agricultural Area.

Channel Hydraulic Characteristics

Manning's roughness coefficient "n" is a major determinant of the hydraulic characteristics of a fluid transport system. For the design of the S-5A system a roughness coefficient of 0.030 was used for the enlarged West Palm Beach Canal (L-10 and L-12). The District made several discharge measurements in the West Palm Beach Canal, and these, together with hydrograph studies of two storm events, were used to obtain an actual value for "n" which was used in this study. The field measurement locations were at Canal Point, near the Big Mound Canal entrance, and at S-5A.

Two methods were applied in the determination of Manning's roughness coefficient from the field discharge measurements. These are "Manning's Method" and the "Method of Velocity Measurement."

Manning's Method

$$n = \frac{1.486}{Q} AR^{2/3} S^{1/2}$$

where n = Manning's roughness coefficient

A = Channel cross-sectional area; in sq. ft.,

R = Hydraulic radius; in feet,

Q = Discharge; in cfs.

S = Energy gradient; in feet.

For each measurement the upstream and downstream channel sections were measured and averaged to obtain the average section for the reach and the average hydraulic radius. Flow (Q) at the upstream and downstream sections was measured and averaged. Energy gradient was determined directly from water surface elevations at the upstream and downstream sections, and was corrected for velocity head.

Method of Velocity Measurement

$$V = 5.75 V_f \log (30Y/K)$$

where V = velocity at any flow location, in ft./sec.

V_f = friction velocity; ft./sec.

g = 32.2 ft./sec./sec.

K = roughness height

Y = depth of flow; in feet.

Manning's "n" can be obtained as follows:

$$n = \frac{(X-1)(Y)^{1/6}}{6.78(X+0.95)}$$

where $X = V_{0.2}/V_{0.8}$

$V_{0.2}$ = velocity at 0.2 depth; ft./sec.

$V_{0.8}$ = velocity at 0.8 depth; ft./sec.

Y = depth of flow; in feet.

A general relation between Manning's "n" and the velocity distribution is shown on Plate 16. This relationship provides the means to estimate "n" when the velocities at 0.2 depth and 0.8 depth, and depth of flow, are known.

The results of application of the two methods to the field measurements made by the District are:

	Manning's "n"	
Method	Reach 1	Reach 2
Manning's	0.026	0.031
Velocity Measurement	0.027	0.030

The computations were based on two discharge measurements made on September 2, 1965. A discharge measurement made on March 27, 1969 was also used for the Velocity Measurement Method determinations.

The values for "n" in Reach 2 (Big Mound to Cross Canal) were checked by hydrograph analysis of the June, 1966 and March, 1970 storm events. Cross Canal inflows were developed using an "n" value of both 0.030 and 0.025 for Reach 2. In the hydrograph analysis, use of "n" = o.025 produced an inflow of 1500 cfs at 10:00AM on July 1, 1966 which compared favorably with a measured value (by the District) of 1430 cfs. A value of 0.025 for Manning's roughness coefficient has been used for the entire West Palm Beach Canal based on an overall evaluation of the data.

A survey of the existing channel (bottom profile and cross-section) was made by the District with its sonic depth finder in April 1971. Cross-sections were spot-verified. The existing bottom profile is shown on Plate 17, together with the design and "as-built" profiles. Channel cross-sections are shown in Appendix A. These actual channel dimensions were used in the analyses made in this study.

Findings

1. The rainfall frequency analysis shows a generally good correlation between rainfall stations within the basin for design storm events or less. A good basis is thereby established for assigning a frequency to storm events to be analyzed.

2. The rainfall-runoff analysis of storm events shows seasonal characteristics for those events of greater frequency than design; "dry" season events being likely to produce less runoff than those of similar magnitude occuring in the "wet" months.

3. Storm events of a magnitude approaching, or greater than, design can be expected to produce the same runoff volume regardless of season of occurrence.

4. Monthly rainfall loss data developed by the ARS for the Everglades Agricultural Area can be applied with reasonable accuracy to the S-5A watershed with some minor adjustment.

5. It can be hypothesized that, on an annual basis, most of the rainfall in excess of 34 inches will quite likely be discharged at S-5A.

6. The hydrologic regime of the S-5A watershed is stable.

7. It is highly probable that for any given storm event up to 10% additional pumping capacity will be available at S-5A because of less than design tailwater conditions in Conservation Area No. 1.

8. Actual pumping experience indicates a long-term pumping requirement less than that estimated in the design phase. This represents a considerable reduction in operating costs and in repair, maintenance and replacement costs when compared with that initially projected.

9. The discharge-duration analysis also strongly indicates the existence of a water conservation feature in the S-5A system when compared to the previously existing uncontrolled condition for the area west of Twenty Mile Bend. Cfs/days of pumping are on the order of 35% less than projected.

10. A Manning's roughness coefficient of 0.025 is applicable to L-10 and L-12.

11. The existing configuration of L-10 and L-12 differs substantially from the design.

ANALYSIS OF STORM EVENTS

The two most severe storm events of reasonably uniform distribution to visit the S-5A watershed were those of June 29-30, 1966 and March 25-26, 1970. See Table 4 for rainfall amounts and estimated frequency of recurrence.

Both events were analyzed using the same procedures. The procedure will be described in some detail for the March 1970 storm event only.

Storm Event; March 25-26, 1970.

This storm has an estimated frequency of occurrence of once-in-ten years. Average rainfall over the basin was 7.52 inches in 29 hours. Rainfall started at about noon on the 25th in the western portion of the basin, and terminated at 5:00PM on the 26th. Rainfall depths were greater over the western end of the basin. Rainfall distribution is shown on Plate 18. Runoff generated by this event was removed in about 12 days; runoff volume approximating 71,000 A. F.

This was the only event since the S-5A system was placed into operation which produced stages at Canal Point in excess of the design stage of 13.7 ft. msl. Canal Point stage was above design for approximately 87 hours. The Canal Point stage hydrograph is shown on Plate 19 together with the stage hydrographs in the West Palm Beach Canal at Big Mound and at the S-5A intake.

Plate 2 represents the state of land development and use at the time of this event. The listing of farm pump capacity on Tables 2 and 3 is indicative of the secondary pumping capacity installed an in operation in March 1970. Total farm pump capacity is 7890 cfs. Secondary pumping station locations are shwon on Plate 20. Plate 21 graphically shows local pumping capacity and location with respect to inflow point on the West Palm Beach Canal.

The storm event was analyzed by deriving discharge hydrographs for the West Palm Beach Canal at two locations; above the junction of the Big Mound

Canal (Reach 1), and above the junction of Cross Canal (Reach 2). Stage records at Canal Point, Big Mound Canal, and S-5A intake were available for this analysis, together with discharge records at S-5A and S-5A(W) (L-8 inflow). With these data the principle of continuity was applied to each reach in accordance with the continuity equation:

$$I_i - O_i = \frac{dS_i}{dt}$$

where I = local inflow rate; in cfs,

O = outflow rate; in cfs,

S = storage volume; in cubic feet,

i = subscript identifying (1 or 2) Reach 1 or Reach 2,

$\frac{dS}{dt}$ = rate of storage change in reach; in cfs.

The outflow rate was computed by using Manning's formula based on the known stages at each end of the reach in question. Hourly intervals were used. Discharge computations were based on the channel sections and the value for "n" determined as described in the previous section.

The discharge hydrographs for the two reaches, derived as indicated above, are shown on Plate 22. Also shown is the discharge hydrograph at S-5A, computed from the ratings described earlier.

Storage change was computed, again at hourly intervals, by use of the stage changes in the reaches in question. Using the above discharges (outflows) and the storage change, inflow rates to Reaches 1 and 2 were computed from the continuity equation. Inflow from the Cross Canal was computed from the difference between discharge at S-5A (adjusted for L-8 inflow) and discharge from Reach 2. Inflow from the area north of the West Palm Beach Canal between Cross Canal and S-5A was taken to be 14% of the above total.

The derived inflow hydrographs for Reach 1, Reaches 1 and 2 combined, and Cross Canal, are shown on Plate 23. Also shown is the L-8 inflow.

Peak local inflow rate into Reach 1 of the West Palm Beach Canal, above Big Mound, amounted to approximately 1420 cfs on March 28. Design inflow to this reach is 2400 cfs and installed secondary drainage capability is about 3500 cfs. Inflow, then, amounted to about 60% of the design inflow and about 40% of the rated secondary drainage capability. Peak outflow from Reach 1 also amounted to approximately 1420 cfs. This means either one of two things: (a) the canal was handling and passing downstream all the excess that the secondary systems were capable of delivering, or (b) that the primary canal was exercising control over local inflow capability.

A comparison of the Reach 1 discharge and inflow hydrographs in relation to the Canal Point stage hydrograph strongly indicates that (a), rather than (b), is the correct interpretation. Inflow remained constant from about the middle of the 2nd day until the middle of the 4th day, whereas stage did not peak until near the end of the 2nd day and started to recede at the beginning of the 4th day. Had the primary canal been controlling local inflow, inflow rates would undoubtedly have been noticeably higher on both the rising and recession sides of the stage hydrograph. Additionally, had the installed secondary capacity, in total, been capable of achieving an inflow rate greater than 1420 cfs, this would have been evidenced by some increase in Canal Point stage during the third day.

Local inflow into the West Palm Beach Canal upstream of Cross Canal reached a peak value of about 3900 cfs on March 27. At this same time discharge from Reach 1 was estimated to be about 1400 cfs. See Plate 22. The peak inflow to Reach 2, between Big Mound and Cross Canals, is therefore estimated to approximate 2500 cfs. This is 210% of both the estimated design inflow of 1200 cfs and the installed secondary pump capacity. See Plate 21.

A considerable portion of the area contributing runoff to Reach 2 of the West Palm Beach Canal is undeveloped. See Plate 2. The only plausible explanations for the high runoff contribution to this reach are that: (a) gravity discharge from the undeveloped lands reached higher than expected values, and (b) some runoff from lands west of Big Mound reached the West Palm Beach Canal east of Big Mound. Neither explanation is capable of quantitative substantiation. However, a check was made of the potential for developing gravity inflow of the indicated magnitude; on the order of 1500 cfs. Inflow culverts installed in this reach under the Project have a combined capacity of 2400 cfs at a head of 0.5 ft. Canal stages compared with natural ground elevations along this reach indicate that heads up to 1.5 feet could have been developed subsequent to the rainfall occurrence. It can be established, then, that both accessibility and gravity head were ample enough to account for the estimated gravity inflow into this reach.

The total peak local inflow of 3900 cfs into the West Palm Beach Canal upstream of Cross Canal represents about 110% of the projected design inflow of 3600 cfs and about 80% of the installed secondary pumping capacity. See Plate 21. As indicated in the preceding paragraphs, approximately 65% of this inflow entered in Reach 2, the downstream reach.

Peak inflow from Cross Canal was estimated to be 1700 cfs. at 3:00PM on March 26. This inflow is 50% greater than the design inflow of 1130 cfs and about 50% of the installed secondary pumping capacity. It is worthy of note that the area tributary to Cross Canal was substantially developed in 1970 and that the then-undeveloped lands adjacent to L-7 had little or no access to the S-5A primary system. The major portion of the Cross Canal inflow can therefore be considered to have been pumped inflow.

The analysis of this storm event demonstrates the fact that runoff from the lower reaches of the basin is removed first, given a reasonably uniform state of secondary drainage development throughout the watershed. Note that Cross Canal inflow peaked at 3:00PM on March 26, Reach 2 inflow on March 27, and Reach 1 inflow at about 10:00AM on March 28.

No quantitative determination can be made as to the respective contributions of pumped and gravity secondary inflow to the approximate 5300 cfs discharge rate at S-5A. However, the analysis indicates a maximum possible pumped inflow, as follows:

Reach		Maximum Possible Pumped Inflow
1		1420 cfs.
2		1200 cfs.
Cross Canal		1700 cfs.
	Total	4320 cfs.

The maximum possible peak pumped inflow would then be about 80% of the total. This compares with the approximately 75% of the watershed which is served by secondary, or farm, pumps.

Pertinent data on pumping operations are shown on Plate 22, and are to be related to the S-5A stage hydrograph on Plate 19. Normal dry season control stage was being held prior to the advent of the storm. Pumping commenced about 5 hours after precipitation started at Canal Point, first with 3 pumps and then with 5. The 6th pump was not put on line at this time since some difficulties were being experienced with over-heating of lubricating oil. When this malfunction was corrected the 6th pump was immediately put on line. Some L-8 inflow was initially pumped. Satisfactory stage was being held at S-5A, and

Canal Point stage had levelled off. When Canal Point stage started to increase, L-8 flows were cut off. No cut-back in engine speed was made until Canal Point stage was definitely in the recession phase. At that point L-8 inflows were again introduced and pumped at S-5A, with a peak discharge of 1600 cfs, for the remainder of the period.

Maximum pumping capability was achieved at about 2:00PM on March 26 and was sustained until midnight on March 28, a period of 58 hours. At the outset of this period S-5A intake stage approximated 12.4 ft. msl. and for 34 hours stage fluctuated between 12.0 ft. msl. and 12.4 ft. msl. Even with maximum pumping no further drawdown could be achieved until local inflow rates subsided. The possibility of achieving the design drawdown stage of 8.30 ft. msl. never existed once the full impact of local inflow had reached S-5A.

Storm Event: June 29, 30, 1966.

This storm has an estimated frequency of occurrence of once-in-four to five years. Average rainfall over the basin was 5.61 inches in 36 hours. Rainfall started at about 6:00AM on June 29 and terminated at about 6:00PM on June 30. Rainfall depths were quite uniform over the entire basin. Rainfall distribution is shown on Plate 24. Runoff generated by this event was removed in about 9 days; runoff volume approximated 43,000 A.F.

Design stage at Canal Point was not exceeded during this event. The Canal Point, Big Mound and S-5A intake stage hydrographs are shown on Plate 19.

No detailed information is available concerning the state of land and secondary drainage development, particularly sugar cane, in the Everglades Agricultural Area took place prior to 1966. Consequently, it is reasonable to assume that the stage of development in 1966 was not less than 80% that of 1970.

Discharge and inflow hydrographs were derived as described for the March 1970 event. The discharge hydrographs are shown on Plate 25 and the inflow hydrographs on Plate 26.

Peak local inflow rate into Reach 1 of the West Palm Beach Canal, above Big Mound, approximated 1500 cfs and occurred at 4:00PM on July 1. This represents about 62% of the design inflow. Peak discharge out of this reach at this time approximated 1400 cfs. Although Big Mound stage shows a slight rise at the time, it can be concluded that the West Palm Beach Canal was satisfactorily handling and passing all, or nearly all, the local inflow coming to it in this reach.

Peak local inflow rate into West Palm Beach Canal above Cross Canal reached a value of 3500 cfs on July 1 at 2:00AM. At this time Reach 1 discharge was 1250 cfs. Therefore, local inflow into Reach 2 is estimated to have been about 2250 cfs, or 85% greater than the projected design inflow.

The total local peak inflow of 3500 cfs into the primary canal upstream of Cross Canal very closely approximated design inflow of 3600 cfs. The inflow distribution between Reach 1 and Reach 2, however, is not in accordance with the design. Again, the greater than design inflow into Reach 2 is probably explainable by the accessibility of undeveloped land drainage to the primary canal and the availability of gravity head due to the favorable position of these lands with respect to S-5A. Peak gravity inflow was quite probably in the range of 1100 to 1300 cfs.

Peak inflow from the Cross Canal, at a rate of 1650 cfs, occurred at 6:00AM on July 1. A mean daily maximum discharge of 1250 cfs was derived. A flow measurement of 1430 cfs was made by the District at 10:00AM on July 1. Extensive flooding occurred in the Cross Canal area during this event, attributable to the fact that Cross Canal had not been enlarged at this time and that the rock weir below the U. S. 441 bridge was still in place. The peak inflow of 1650 cfs is about 96% greater than the design inflow rate for the unimproved Cross Canal.

During this event peak inflow from Cross Canal came subsequent to the peak inflow from Reach 2. This is explained by the effect of the rock weir which both delayed discharge from Cross Canal and created high (flooding) stages in Cross Canal. The rapid recession of the Cross Canal inflow hydrograph is indicative of quick release of both accumulated canal and on-the-land storage once the controlling feature of the rock weir had been overcome.

Information pertinent to pumping operations during this event is shown on Plate 25. Pump operations started immediately with the incidence of rainfall and at full capacity. A comparatively rapid drawdown of 2.5 feet, to an intake stage of 9.5 feet msl., was achieved. L-8 inflow was being accepted and pumped from the outset. Pumps were cut back while still maintaining a stage of 9.5 ft. msl. A second high intensity rainfall then occurred, intake stages rose, L-8 inflow was cut off and all pumps were placed back on line. By this time, intake stage rose to 10.5 ft. Full capacity pumping was sustained for over 60 hours. During this period intake stage fluctuated between 10.0 ft. msl. and 10.5 ft. msl. As in 1970, no further drawdown could be obtained once maximum pumping rates were achieved. L-8 inflows were again introduced once Canal Point stage firmly evidenced recession.

<u>Comparison and Summary of the Two Storm Events</u>

Pertinent data for the two storm events are summarized in the following tabulations:

	June 1966	March 1970
Rainfall	5.61"/36 hrs.	7.52"/29 hrs.
Runoff (inches)	3.69"	5.78"
Runoff (A.F.)	43,000 A.F.	71,000 A.F.
Runoff removal time	9 days	12 days
Runoff removal rate (average)	4800 A.F./day	5900 A.F./day
Peak stage, Canal Point	13.5 ft. msl.	14.8 ft. msl.
Peak stage, Big Mound	12.5 ft. msl.	14.2 ft. msl.
Sustained drawdown, S-5A	10.0-10.5 ft. msl.	12.0-12.4 ft. msl.
Peak inflow, Reach 1	1500 cfs.	1420 cfs.
Peak inflow, Reach 2	2250 cfs.	2500 cfs.
Peak inflow, Reach 1 & Reach 2	3500 cfs.	3900 cfs.
Peak inflow, Cross Canal	1650 cfs.	1700 cfs.
Max. sustained discharge,S-5A(cfs)	5150 cfs.	5250 cfs.
Max. daily discharge, S-5A(inches)	0.78"	0.95"

The striking feature of this comparison is the closeness of the peak inflow values for each reach of the West Palm Beach Canal and the Cross Canal for two storm events of substantially different intensities. This demonstrates convincingly that inflow is rather firmly regulated and performance governed by some combination of the design and condition of the system, system operation, and secondary inflow structural measures.

The basic feature controlling performance of the system is the pumping capacity at S-5A. Closely related to this is the gravity head which can be developed in the system by drawdown at S-5A. Water was being moved through the system at about the same rates in 1966 as in 1970, but at lower stages.

Therefore, the velocity of water movement was somewhat greater in 1966 than in 1970 because steeper gradients had been developed.

The inflow and discharge hydrographs for both events show a balancing of local inflow into each reach with discharge from the reach, at approximately the same numerical quantities for both events. There was no stage increase at either Canal Point or Big Mound during the period of maximum discharge at S-5A for either event. Had there been excessively high rates of inflow beyond the capability of the West Palm Beach Canal to carry, Canal Point and Big Mound stages would have risen during this period of maximum pumping. This was not the case. Accordingly, the conclusion can be drawn that conditions external to the primary system itself which can affect and control inflow to Reaches 1 and 2, and from Cross Canal, were approximately the same for both events despite the difference in total rainfall amounts.

Considering local inflow rates, then, both events were very nearly identical in terms of their impact on the primary system. The primary system itself did not significantly control inflow rates. The difference in total rainfall amount and intensity was expressed primarily in terms of the duration of local inflow rather than the rate of inflow. If this is a correct assumption the primary system should have performed about the same in all respects in 1970 as it did in 1966. This was not the case, since stages in West Palm Beach Canal were 1.3 to 2.0 feet higher in 1970 than in 1966. The stage differences are not completely accounted for by the slightly greater volumes of water being discharged out of Reach 2 in 1970. Since very nearly the same maximum pumping rates were achieved in both events, the different performance in 1970 may well have been influenced by the differences in the drawdown feature.

The system performed satisfactorily, exclusive of Cross Canal, under the design condition (June 1966 event). Design stage at Canal Point was closely approximated. Flow rate in the West Palm Beach Canal upstream of Cross Canal closely approached the design value. Peak Cross Canal inflow exceeded the design inflow of the unimproved channel by about 96% whereas flow rates in West Palm Beach Canal upstream of Big Mound Canal were only 62% of the design rates. Throughout almost the entire runoff period S-5A removed surplus water from the L-8 area at a daily rate of 700 cfs to 1200 cfs. A pumping rate at S-5A of about 5150 cfs, nearly 10% over design, was sustained for about 3 1/2 days. This additional pumping capability resulted from a tailwater stage lower than design in C. A. #1.

Under the 10-year storm event (March 1970) the system still performed reasonably satisfactorily. Design stage at Canal Point was exceeded by about 1.1 ft., and stage remained above the 13.7 ft. design stage for a period of 4 days. Peak flow rates in the West Palm Beach Canal in the reach upstream of Big Mound approximated 60% of the design; and in the reach upstream of Cross Canal were about 10% above design rates. Peak Cross Canal inflow was about 50% above design. Runoff from the L-8 area at rates between 500 and 1500 cfs was removed once Canal Point stages had receded to design stage. Pumping rates in excess of the design rate of 4800 cfs were sustained at S-5A for a period of about 3 days.

Findings:

1. Design conditions in terms of inflow rates, basin discharge and stages were generally attained during the design event (June 1966). These conditions, however, resulted with the basin in a state of less than full development.

2. During the 1966 event, L-8 inflows were almost continuously pumped for the entire runoff period. This capability could be applied to the S-5A watershed. In view of this it is probable that with a recurrence of the design event under the present state of development the S-5A primary system would perform as satisfactorily as it did in 1966.

3. The primary system responded satisfactorily to the 10-year storm event (1970), evidencing a degree of expanded capability in terms of runoff removal. This, however, represents the probable maximum system capability and, in turn, is dependent to some degree on stages in Conservation Area No. 1.

4. The primary system is now at about the maximum limit of its design capability, with little reserve remaining in the system, and with about 25% of the watershed area not fully developed in terms of improved secondary drainage (farm pumps).

5. Features affecting and controlling the future performance of the S-5A primary system can be identified as follows:

(a) Condition of the primary system.

(b) Operation of the primary system.

(c) Regulation of secondary inflow.

Each of the above features was examined as a part of this study, and are discussed in the following sections.

CONDITION OF THE PRIMARY SYSTEM

The primary system is considered to be S-5A and the West Palm Beach Canal (L-10 and L-12).

The head conditions under which S-5A can be expected to operate were discussed in a previous section. That discussion indicated the high probability that for any single storm event, operating heads will be lower than design and hence that a runoff removal capability somewhat greater than design will be available.

Pumps and pumping equipment are in excellent condition under a rigorous maintenance, repair and parts replacement program. Maximum engine speed has been found to be 720 rpm as against the design of 714 rpm. A slightly greater discharge capability therefore exists for all heads.

The S-5A portion of the primary system can be expected to remain as effective in the future as it is now, and as it was under the two major storm events analyzed in this study.

The West Palm Beach Canal (L-10 and L-12) channel was excavated to the design bottom profile shown on Plate 17 and to the sections indicated in Appendix A. Plate 17 and the Appendix also show the channel condition as of 1971. These show that there has been a substantial deposition of material on the bottom of the primary channel since completion of enlargement in 1956.

Spot probings of the channel were made in May 1973 at 6 locations to verify the data obtained by the sonic depth finder in 1971. The probings substantiate the existence of depositional material on the channel bottom to depths up to 6-7 feet, as shown on Plate 17. The probings indicate that the deposited material is organic in nature, probably having settled out from farm pump discharges and land runoff over the past 17 years. It is noted that the greatest depths of deposit are adjacent to the Cross Canal entrance.

The effect of the modified channel section was examined by deriving a water surface profile for the design flow condition. Actual channel sections and the established value of 0.025 for Manning's "n" were used. The District's backwater computer program was used for this analysis. The derived water surface profile is shown on Plate 17.

Also shown on Plate 17 are water surface profiles for the design flow condition in the design channel using values for "n" of 0.025 and 0.030. The Corps of Engineers' design was based on an assumed value of "n" equal to 0.030.

The actual water surface profile (existing channel section and "n" = 0.025) approximates for all practical purposes the design water surface profile (design channel and "n" = 0.030). The reduced actual section is balanced by the improved actual roughness coefficient. Consequently, actual canal performance under design conditions would be about the same as projected in the system design. This, of course, is borne out by the analysis of the June 1966 storm event.

However, this equivalence of performance will result only as long as no further deposition takes place. There is no reason to believe that this will be the case in the future.

The backwater computations indicate that a reduction in design stage of at least one foot at Canal Point and Big Mound will result if the design canal section is restored. System performance in the future would thereby be enhanced. This is also an important consideration in terms of local water control costs as the continued subsidence of the adjacent agricultural mucklands is taken into account.

OPERATION OF THE PRIMARY SYSTEM

The general pumping operations procedure was outlined earlier in this report. See page 6. The application of this general procedure to the two specific storm events analyzed is summarized on pages 21 and 24 with pertinent data given on Plates 22 and 25. An indication of the possible importance of the early drawdown feature of pumping operations to overall system performance was given in the discussion on pages 25 and 26.

As noted in the earlier discussions herein, it is apparent that once maximum discharge capacity is reached at S-5A and inflow is reaching the pump intakes at rates sufficient to sustain that discharge, the pumps are incapable of accomplishing any further intake stage drawdown. Water surface gradients and resultant flow velocities throughout the period of peak runoff removal are thus largely governed by the state of the primary system in these respects at the time peak pumping rates are reached. On the other hand, the performance of the system in the early portion of the June 1966 event convincingly shows that significant drawdown can be accomplished before the impact of local inflows reaches S-5A.

The analysis of the two storm events seems to indicate that overall performance should have been about the same under both conditions. It was not, since Canal stages throughout were higher in 1970. A possible explanation for this lies in the fact that in 1966 an immediate drawdown was created by putting all pumps on line at full capacity when precipitation started, whereas in 1970 only 3 pumps were intially placed in operation and then when about 3'' of rain had already fallen at Canal Point.

The chronology of events during the 1970 occurrence reveals the manner in which specific conditions surrounding a storm event can affect performance.

Such conditions, however, are not necessarily unusual. Without having received notification of nearly 3 inches of rain having fallen at Canal Point, S-5A was secured at 4:30PM on March 25. At that time less than 1 inch of precipitation had been measured at S-5A. When notification was received that stage at S-5A had risen (6:00PM), pumping operations were started (7:15PM), putting three pumps on line, with two more pumps being put on at 10:30PM. The 6th pump, as noted earlier, could not be placed on line due to malfunction in the engine lubricating oil system. The specific conditions which affected performance in this instance were: (a) lack of information concerning the situation throughout the watershed, and (b) initial lack of full pumping capability.

With proper information and full pumping capability available, it appears that a drawdown quite similar to that of 1966 could have been attained in 1970, since Cross Canal and Reach 2 inflows were no greater in the first few hours of the 1970 event than there were in 1966. Once the initial drawdown had been created it is quite likely that the initial slopes and velocities thus created could have been generally maintained, just as they were in 1966.

This indicates that any delay in accomplishing an early drawdown at S-5A should be avoided in the future. It also indicates that, if at all possible, drawdown should start whenever significant rainfall is imminent. Local agricultural interests usually seem to have no hesitancy about releasing stored water in their systems if rainfall appears to be likely. District pumping operations should be geared to this procedure whenever practicable. An "early-warning" system of notification by major agricultural operators has been developed to this end. It should be adhered to since, obviously, the major beneficiaries are the agricultural operators themselves.

There could be occasions of "false starts" under this type of procedure. This should not be a deterrent. The worst that could happen would be the loss of canal storage. This is a comparatively small volume and is normally readily replaceable by withdrawal from Lake Okeechobee.

An early drawdown will be of most benefit to the lands west of Big Mound. The early lowering of stages near S-5A will remove Cross Canal storage and inflows somewhat more quickly, thus increasing the gradient and rate of water movement between Canal Point and S-5A.

Lower intake stages at S-5A will slightly reduce discharge capacity due to the increased pumping head. This can be compensated for by lengthening the duration of pumping at maximum capacity.

Overall system performance in terms of Canal Point stage could be improved by longer duration pumping at maximum capacity and/or delaying the acceptance of inflow from L-8 at S-5A. As described on page 6, pumping is generally cut back, or L-8 inflows are accepted, once Canal Point stage firmly evidences recession. If a stage of 12.5 ft. msl. at Canal Point can be considered the optimum, performance in this respect would be improved if no cut-backs were made (except if drawdown stage went below the design stage of 8.3 ft. msl.), or L-8 inflow accepted, until this stage was reached. In this situation, however, it is understood that some judgment might still have to be exercised in balancing conditions at Canal Point against those in L-8 existing at the time.

REGULATION OF SECONDARY INFLOW

Derivation of Everglades Runoff Formula

Allowable local inflow is allocated by the District in accordance with the Everglades Runoff Formula. In general terms, this provides for a higher allowable unit runoff rate (inches per day, or CSM) for smaller tracts of land. The rationale for this type of treatment rests on the fact that the length of time it takes for peak runoff to reach the point of discharge (time of concentration) is shorter for a small tract when compared with a large tract. Thus the contribution of runoff from smaller tracts to peak flow in the primary channel is, theoretically, less than that from larger tracts.

The Everglades Runoff Formula was derived for application to the entire Everglades Agricultural Area of approximately 1100 sq. mi. The formula in its present form is expressed as follows:

$$Q = \left(\frac{81}{\sqrt{A}} + 13 \right) A$$

where: Q = Allowable inflow; in cfs.

A = Area of tract producing runoff; in sq. mi.

Note: throughout this section "Q" is expressed in cfs, and "A" in sq. mi.

The derivation of this formula is based on:

1. A 24-hour rainfall having a once-in-five year frequency, i.e., 6.5 inches.
2. Rainfall excess of 3.5 inches.

Two fixed points on the curve are established as representing:

1. Allowable runoff for the entire watershed area based on the Project design of 3/4 inch per day runoff removal; or 20 CSM.*

2. Allowable runoff from a one square mile tract equivalent to complete removal of rainfall excess from the design storm; i.e., 3.5 inches per day, or 94 CSM.

* Entire watershed area taken to be 140 sq. mi. in this derivation since formula is applicable to all basins in the Everglades Agricultural Area; 140 sq. mi. being generally representative of basin size.

The curve based on this formula is shown on Plate 27.

Evaluation of Everglades Runoff Formula

The capacities of the farm pumps is an important factor in the investigation of basin runoff. This evaluation of the Everglades Runoff Formula started with an examination of existing installed pump capacity in relation to the formula being used by the District. The capacities of existing farm pumps are plotted against the size of their service areas on Plate 27, in relation to the Everglades Runoff Formula curve.

A number of the installed pumps lie above the Everglades Runoff Curve. This is because many of the pumps in this watershed were installed prior to District application of the allowable inflow formula. However, for smaller tracts in particular (640 acres and less) all of the existing installations lie below the curve. This indicates that the formula is not in conformity with what local operators believe to be economical in terms of pumping capacity required to remove runoff from smaller-sized tracts. The average installed pumping capacity for one square mile (640 acres) tracts is 55 CSM, or about 2 inches of runoff per day.

As very general confirmation of reasonableness of this lower value, it is noted that the Everglades Experiment Station has recommended a runoff removal rate of 2 to 3 inhces per day for truck crops and 1 inch per day for sugar cane and pasture.

A tentative revision of this formula can be derived on this new basis by using the hydrologic data developed earlier in this current investigation, as follows:

1. Twenty-four hour rainfall of once-in-five year frequency equal to 5.0 inches; see Plate 4.
2. Rainfall excess of 3.4 inches; see Plate 6, Figure A.
3. Peak discharge from 221 sq. mi. watershed area equal to 19.4 CSM; see Plate 15.
4. Peak allowable discharge from one square mile area equal to 55 CSM.

The result of this derivation is expressed as follows:

$$Q = \left(\frac{38.2}{\sqrt{A}} + 16.8 \right) A$$

This formula is plotted in the form of a curve on Plate 27, and is designated "New Everglades Runoff Curve." Table 6 is a comparison of the existing and "new curve" runoff values.

Land Use - Runoff Relations

The Everglades Experiment Station had recommended, as noted above, different runoff removal capabilities dependent upon land use. Consequently, the relationships between land use and installed pumping capacity in this watershed were examined. It is felt desirable to recognize actual practice wherever possible since it usually reflects practical consideration of economic factors and values.

Plate 28 plots the capacities and service areas for farm pumps serving sugar cane acreage in the watershed. Location identifying designations are given. Plate 29 is a similar plotting for improved pasture acreage and Plate 30 for truck crop acreage.

Runoff equations were derived for each type of land use, as follows:

Sugar cane:	$Q = 55.0A^{0.861}$
Improved pasture:	$Q = 58.0A^{0.801}$
*Truck crops:	$Q = 63.0A^{1.350}$

*This equation is valid only for areas less than 2.5 sq. mi.

The analysis shows that the requirements, as reflected by actual practice, of three major land use types are not greatly different. Runoff values from one square mile are 55 CSM, 58 CSM, and 63 CSM respectively for sugar cane, improved pasture and truck crops. A comparison of these formulae is shown on Table 7. This analysis confirms that it is reasonable to continue to apply a single runoff formula to all lands in the basin regardless of land use.

Local Inflow Analysis

To evaluate local inflow regulation alternatives a hypothetical situation was first assumed wherein all existing pumps were limited to a maximum capacity of 3/4 inch per day. This hypothetical situation was examined for the March 1970 event only since no information is available as to the installed farm pump capacity which existed in June 1966.

For this condition the accumulated inflow above the Cross Canal junction would be 2060 cfs. For the analysis daily hydrographs rather than hourly were used and the following assumptions made:

1. Pumped inflow is removed more quickly than gravity inflow.
2. The recession portion of the hydrograph is the same as for the actual 1970 condition.

3. S-5A pumps respond as under the actual 1970 condition.

4. Total volume of runoff removed is the same as for the actual 1970 condition.

The backwater profile based on 2060 cfs pumped inflow and an assumed gravity inflow of 500 cfs indicates a maximum Canal Point stage of 13.0 ft. msl. A portion of the gravity inflow will therefore be removed along with pumped inflow by S-5A.

A synthetic inflow hydrograph for the West Palm Beach Canal above Cross Canal (Reach 1 plus Reach 2) was derived as follows: (Refer to Table 8).

1. A total of 15,870 cfs/days was computed as the volume of runoff removed by local pumps. A period of removal of 8 days was computed.

2. Hydrograph recession was started after the last day of pumped inflow. Runoff removal during this period was computed to be 7500 cfs/days.

3. Total runoff removal from Reaches 1 and 2 was 26,120 cfs/days. Gravity runoff of 2,750 cfs/days (26,120 - 15,870 - 7,500) was distributed over the first nine days as 300 cfs/day.

The derived inflow hydrograph is shown on Plate 31, using the values in Column 4 of Table 8.

A mass inflow curve was computed from the derived inflow hydrograph and an accumulated discharge curve developed. The discharge hydrograph is plotted on Plate 31 from the values given in the last column of Table 8.

For this hypothetical condition Cross Canal inflow and daily discharge was derived using the same method. Plate 32 shows the hydrographs, using the values listed in Table 9.

The daily discharge hydrograph at S-5A for the hypothetical case was obtained by adding the synthetic hydrographs for Reach 1 plus Reach 2 discharge and Cross Canal discharge, and as assumed gravity inflow from the area on the north of West Palm Beach Canal below Cross Canal. The resultant discharge hydrograph is plotted on Plate 33 together with the actual discharge hydrograph. The actual 1970 discharge hydrograph was developed from the data listed on Tables 10 and 11.

A comparison of these hydrographs shows a significantly lower discharge rate at S-5A (about 30% below available capacity) and a longer duration for runoff removal. It definitely indicates that limiting allowable local runoff to the overall basin runoff value of 3/4 inch per day is not justifiable since there would have been unused, and unusable, primary pumping capacity. Therefore, this is not an acceptable local runoff regulation alternative.

This analysis also demonstrates that some form of allowable inflow curve which recognizes the "time of concentration" factor is a logical and reasonable approach to local inflow regulation.

Projected Future Development of Basin

The 1966 and 1970 storm events were analyzed on the basis of their recurrence at some future date when secondary drainage systems are completely developed (pumped) and all land is placed into agricultural use.

This requires a projection of future pump locations and capacities for the undeveloped lands. The area not now under farm pump water control approximates 61 sq. mi., with most of it located in the downstream portion of the watershed.

For this analysis inflow to Reach 1 was assumed to remain unchanged. Land ownership information was used to locate new pumping installations and the present Everglades Runoff Formula was applied to establish new pumping capacities. Inflow and discharge hydrographs for this condition for the March 1970 event were derived as described in the previous section. The data is listed in Table 12 and the S-5A discharge hydrograph is plotted on Plate 34. Maximum S-5A pumping capacity is set at about 5300 cfs. The actual March 1970 hydrograph is shown on Plate 34 for comparison.

A backwater computation shows that Canal Point stage for this condition will be about one foot higher than stage under present conditions of land development and secondary drainage improvement. Storage in the primary canal will be increased (expressed as higher canal stages) as will storage of runoff on the land (flooding).

A similar analysis was made of the June 1966 event. Even though this watershed is substantially developed, these analyses indicate that future system performance can be improved by modifying the present allowable local inflow formula for application to future secondary improvements.

Another factor to be considered here is the essentially unregulated gravity inflow now entering West Palm Beach Canal from the undeveloped lands in the lower portion of the basin. See discussion on page 22. Placing this runoff under regulation by appropriate sizing of pumping capacities may well improve the flood discharge regime of the S-5A primary system.

An Everglades Agricultural Inflow Curve

More than half the existing pumps in this basin have a capacity greater than the "new" Everglades Runoff Curve developed earlier herein. The existing pump data together with the synthetic hydrograph analyses discussed above were used to obtain a drainage area - discharge relationship. This relationship is plotted on Plate 35.

A curve fitting these data points was developed and is expressed as follows:

$$Q = 60.5A^{0.80}$$

This is compared with the existing Everglades Runoff Formula in Table 13, and the curve is plotted on Plate 27.

This curve fits both existing and projected conditions somewhat better than the "new" Everglades Runoff Curve and represents a reasonable balance of all pertinent factors. In the application of this curve the maximum allowable unit runoff removal rate would be 60.5 CSM (2.2"/day) for all tracts 640 acres or less in size.

Cross Canal and Gladeview Canal Inflow

Table 3, which lists presently installed secondary pumping capacity on Cross Canal, and the map of Plate 20, show a large number of pumping installations serving the Gladeview Drainage District. These local pumps discharge into Gladeview Canal which, in turn, discharges through the S.R. 80 box culvert into Cross Canal.

Runoff values from Cross Canal derived in this report indicata a maximum of 1650 cfs for the June 29-30, 1966 event, and a mean daily maximum of 1250 cfs (page 25); these derived values having been verified by an actual discharge measurement. It also indicates for the March 25-26, 1970 event a maximum discharge of 1700 cfs, and a mean daily maximum of 1330 cfs. The unit runoff for the 1966 event was 0.76 inches compared with 0.77 inches in the remainder of the basin, while for the 1970 event it was 0.81 inches compared with 0.94 inches.

The Cross Canal basin contains 61.0 square miles, or 27 percent of the S-5A basin. The total installed farm pump capacity is approximately 3340 cfs, providing a runoff capability of 2.04 inches per day. This compares with the overall capability of 1.94 inches per day for the 160 square miles of developed land in the entire S-5A watershed.

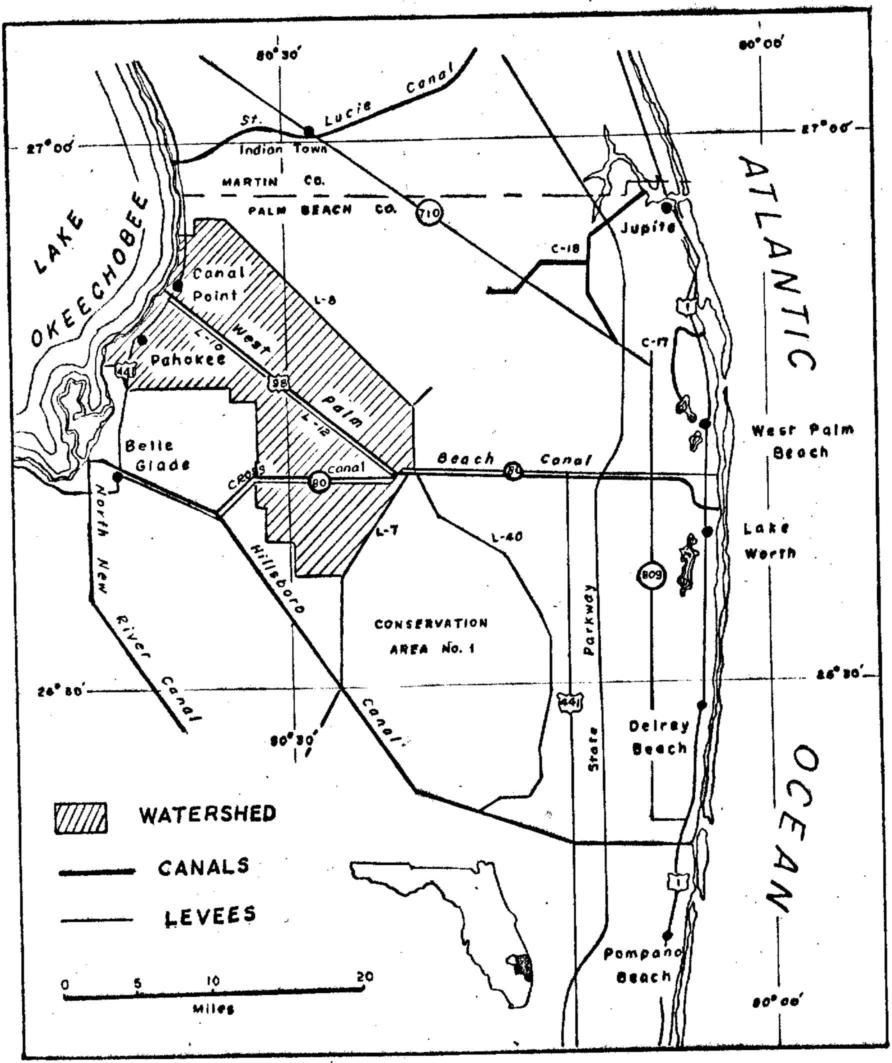


PLATE 1 LOCATION OF STUDY AREA

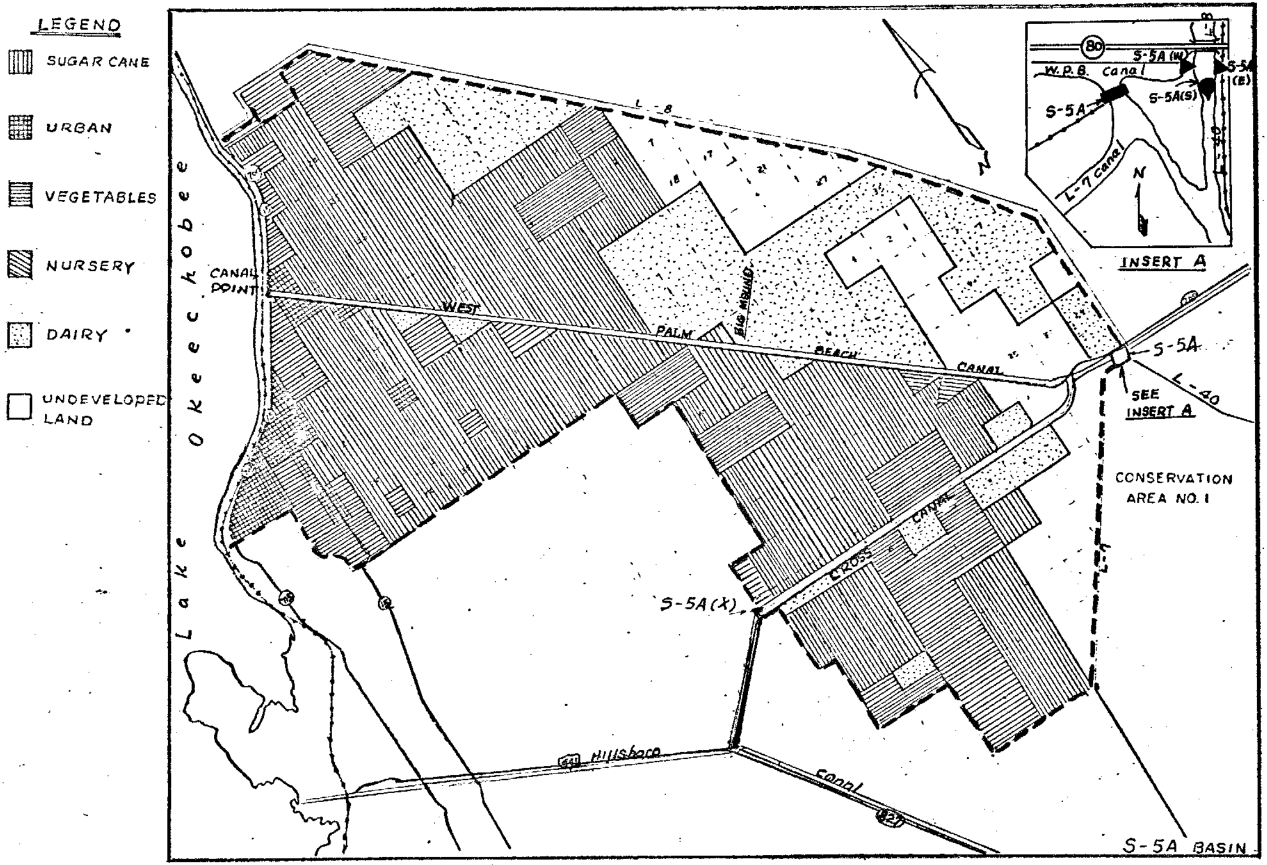


PLATE 2 LAND USE AS OF JANUARY 1970

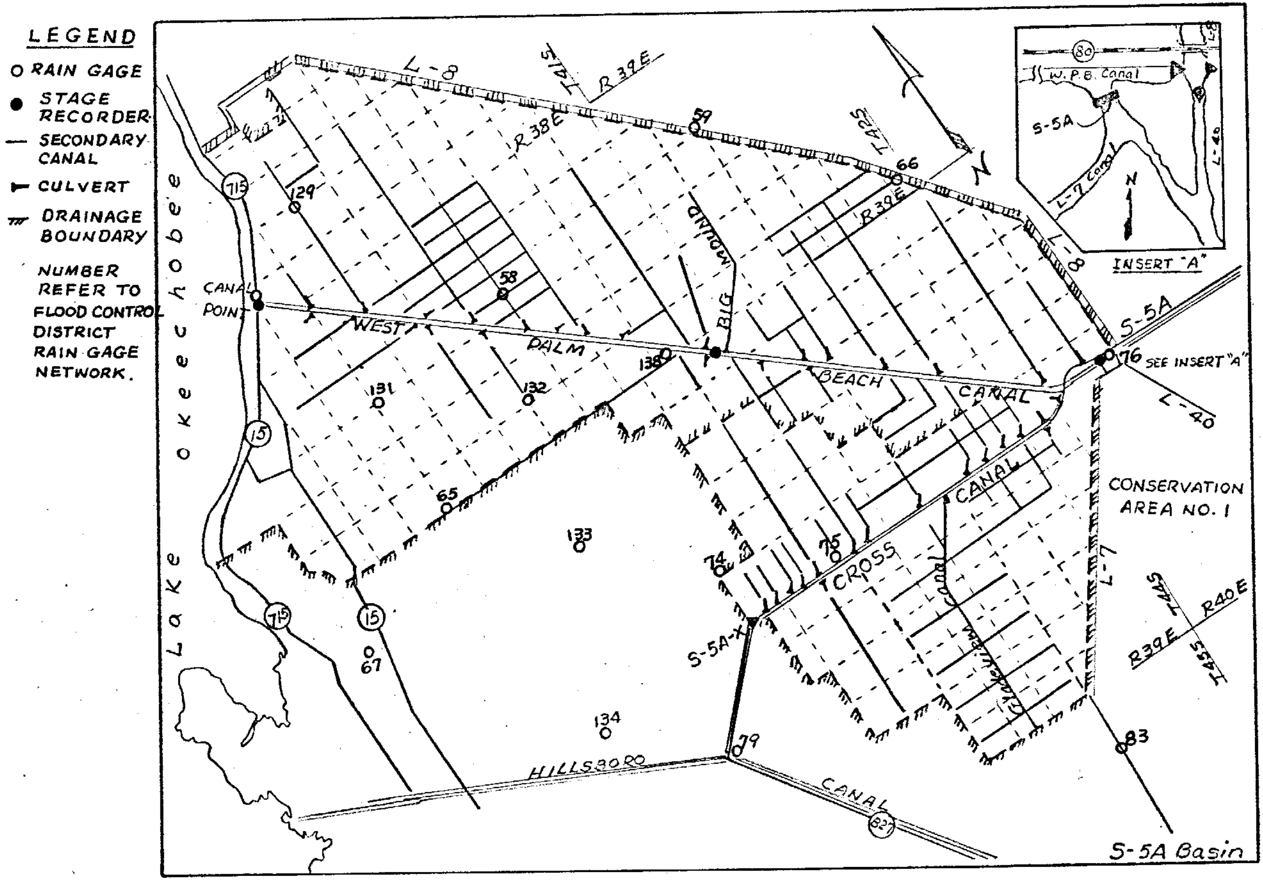


PLATE 3 SECONDARY DRAINAGE SYSTEM WITH STAGE AND RAINFALL STATIONS

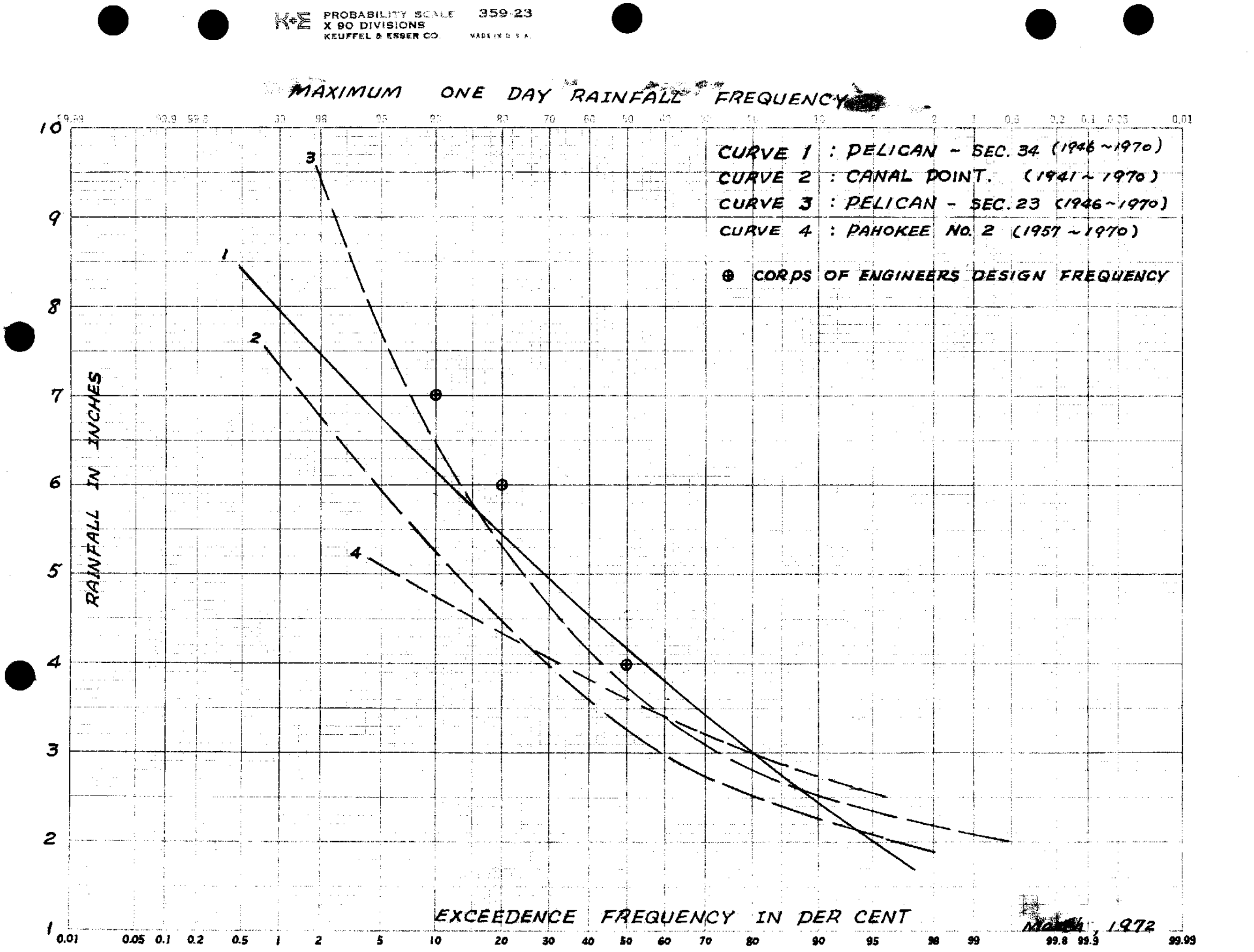
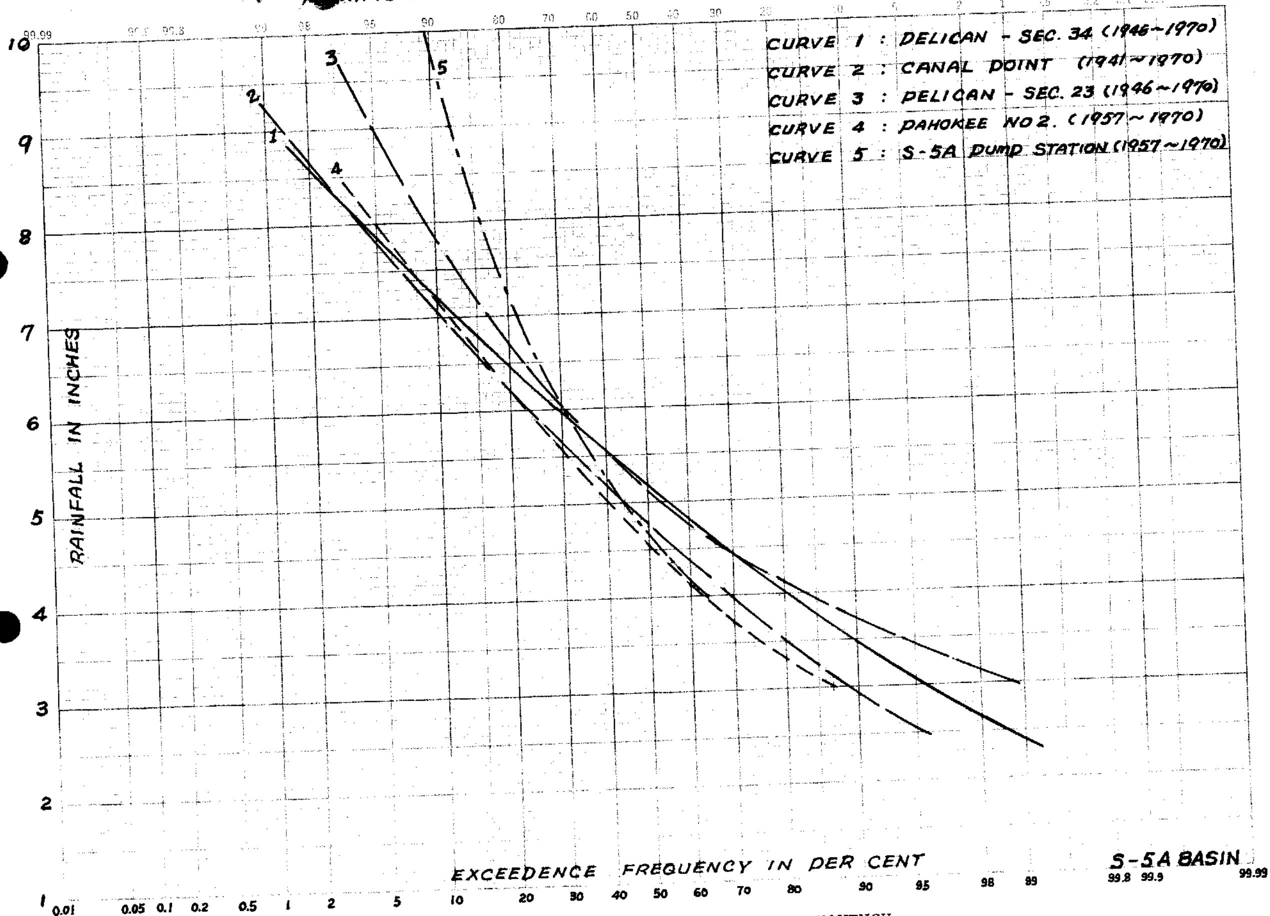


PLATE 4. MAXIMUM ONE-DAY RAINFALL FREQUENCY

K+E PROBABILITY SCALE X 90 DIVISIONS KEUFFEL & ESSER CO. 359-23 MADE IN U.S.A.

MAXIMUM TWO DAYS RAINFALL FREQUENCY



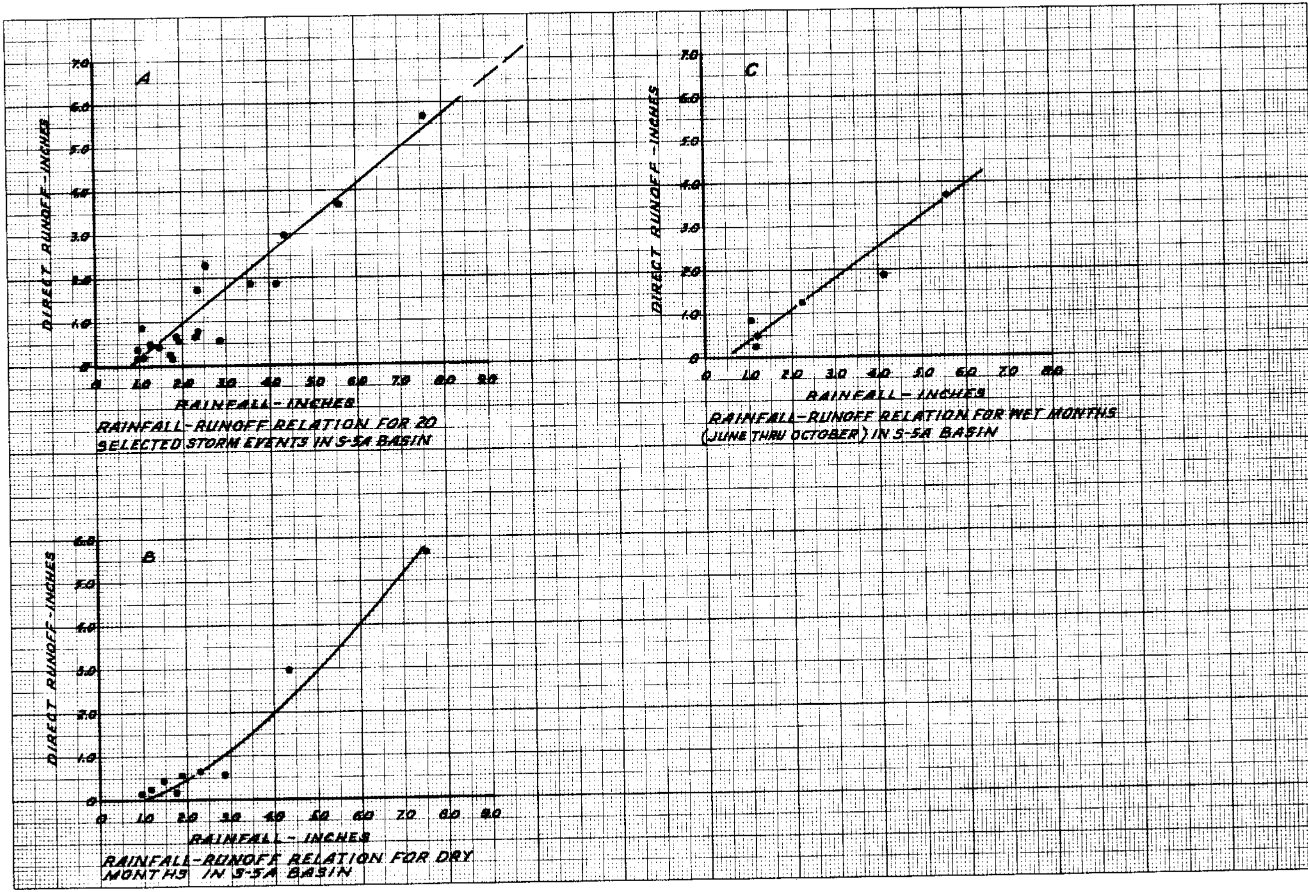


PLATE 6 - RAINFALL-RUNOFF RELATION OF TWENTY STORM EVENTS

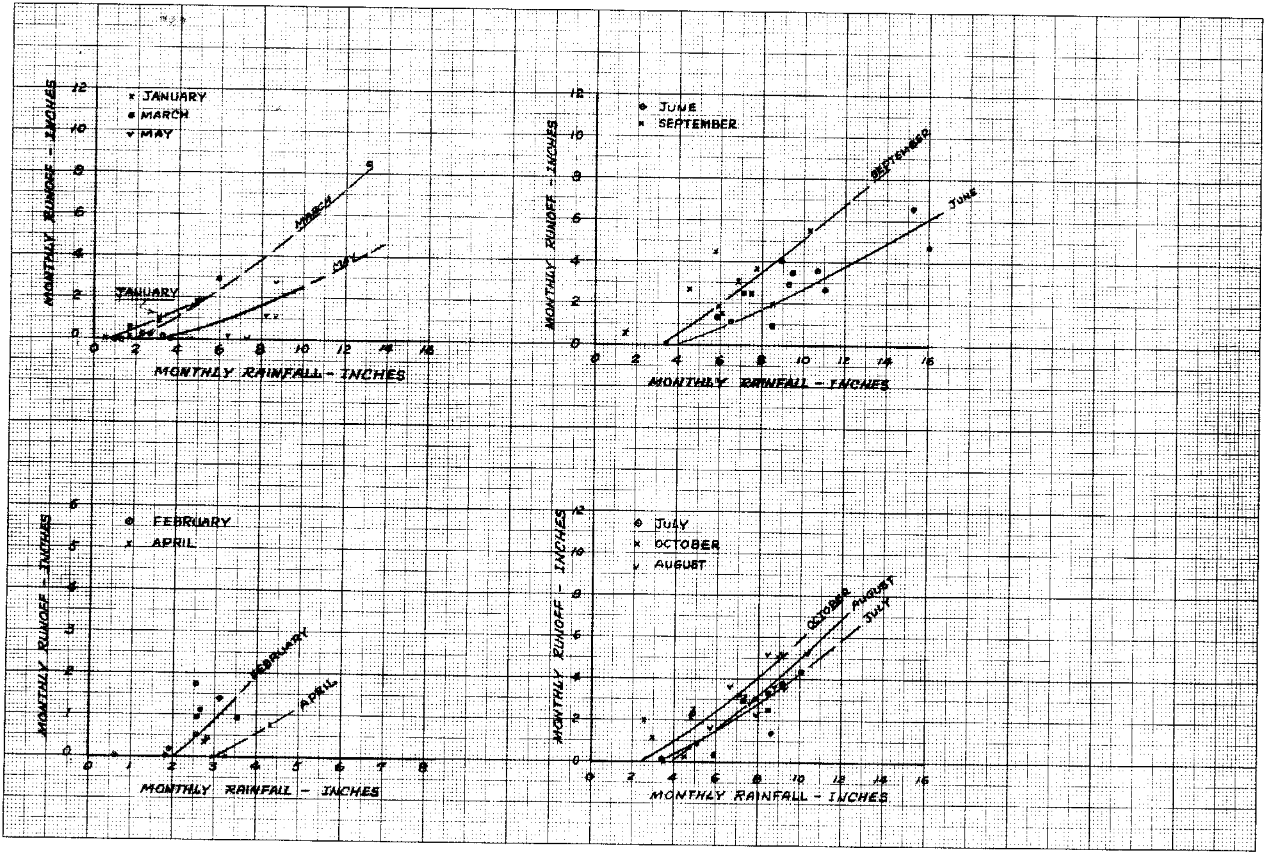


PLATE 7. MONTHLY RAINFALL-RUNOFF RELATIONS

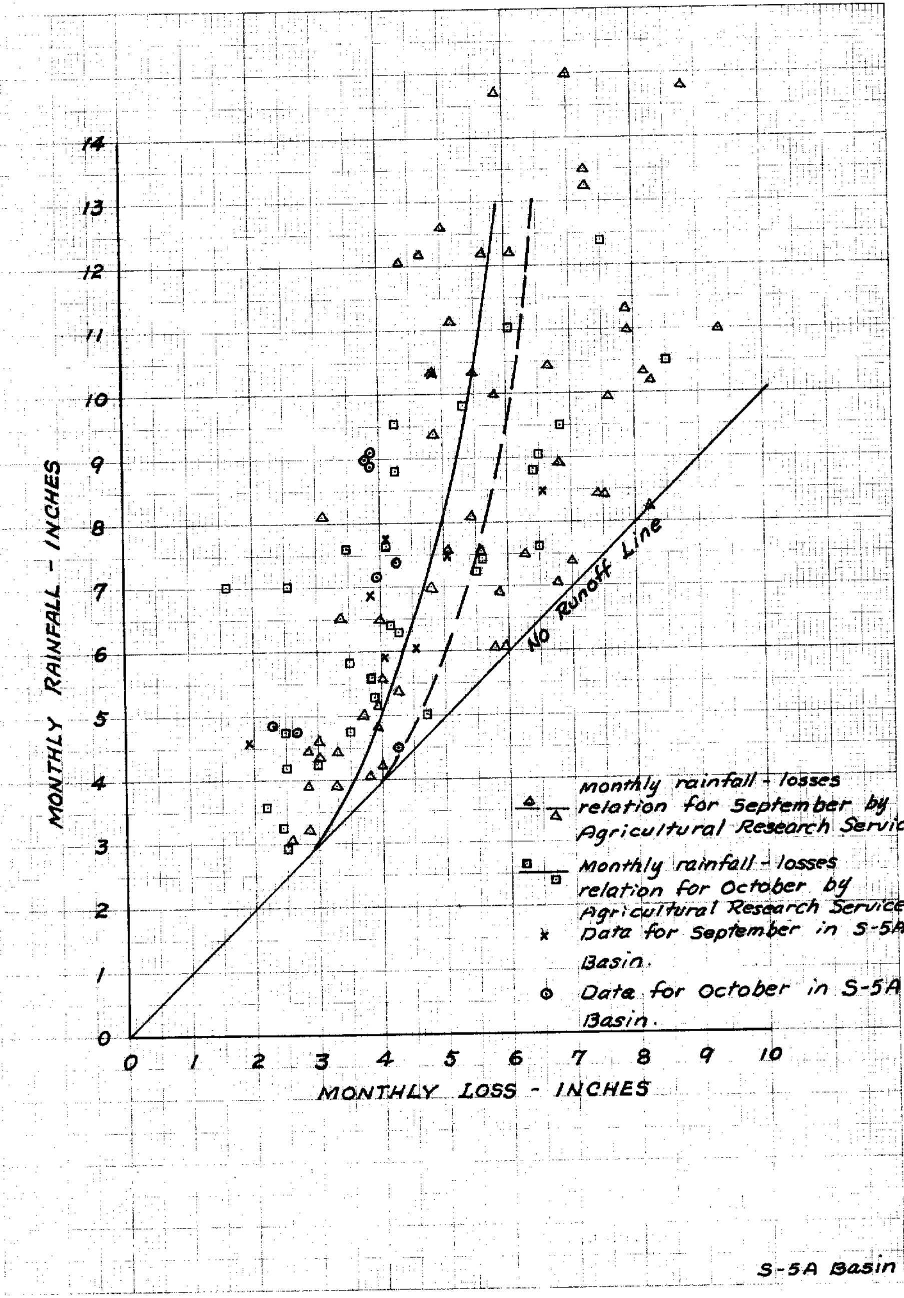


PLATE 9. RAINFALL-EXCESS EVALUATION FOR SEPTEMBER AND OCTOBER AS COMPARED TO

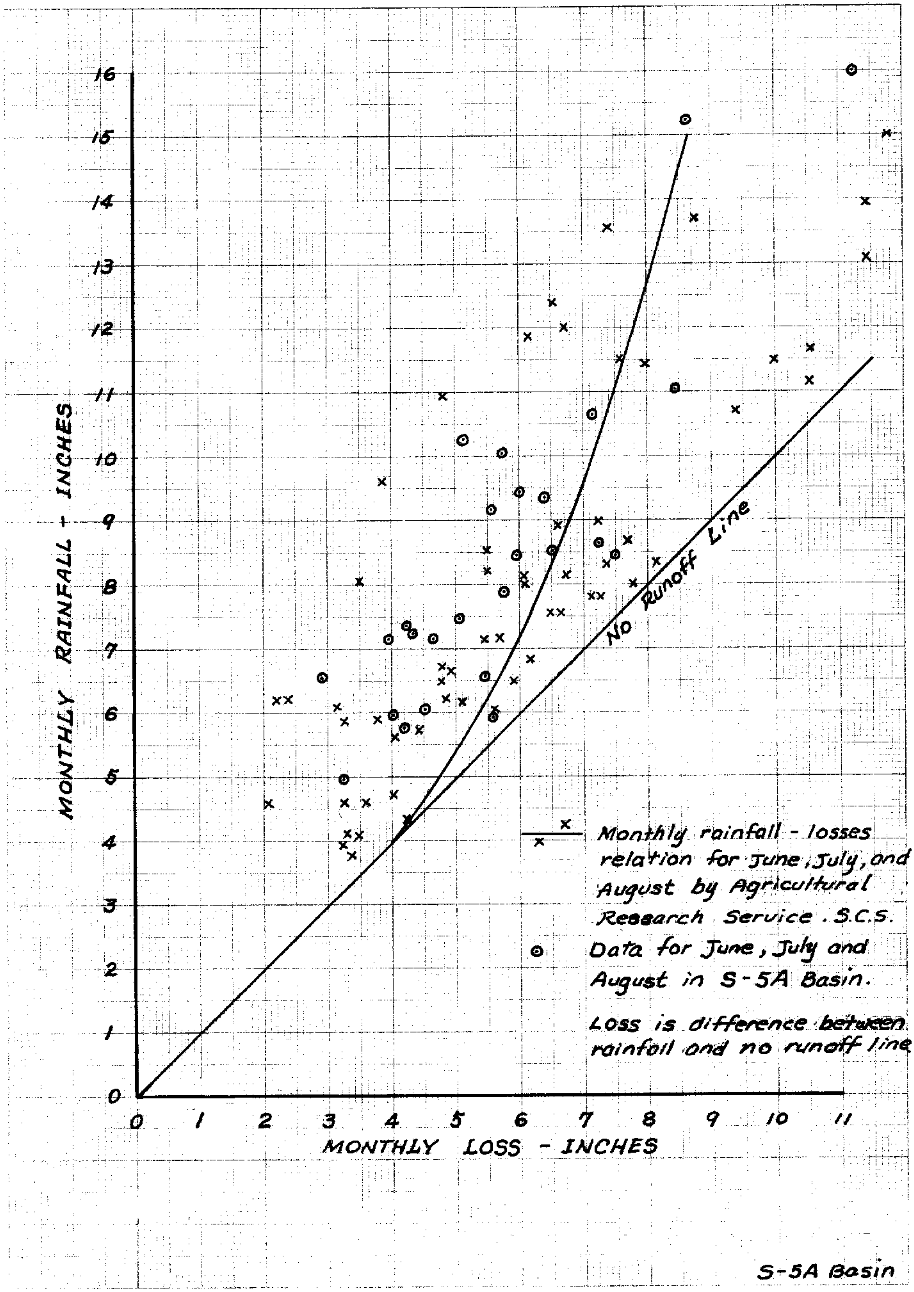
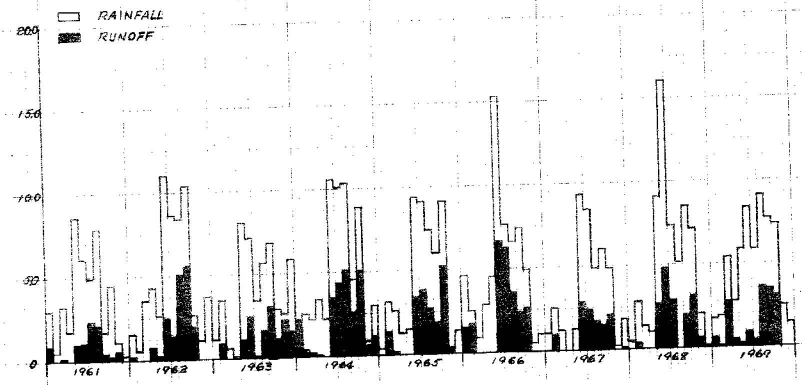


PLATE 8. RAINFALL-EXCESS EVALUATION FOR JUNE, JULY, AND AUGUST, AS COMPARED TO THE RELATION DEVELOPED BY AGRICULTURAL RESEARCH SERVICE, SCS

INCHES

Annual Rainfall	43.70	58.71	48.22	61.12	51.64	59.01	41.70	60.50	61.38
Runoff	8.97	17.81	14.10	27.04	19.30	24.87	11.71	16.69	15.66
Rainoff in excess of 34 inches	9.70	24.71	14.22	27.12	17.64	25.01	7.71	26.50	27.35



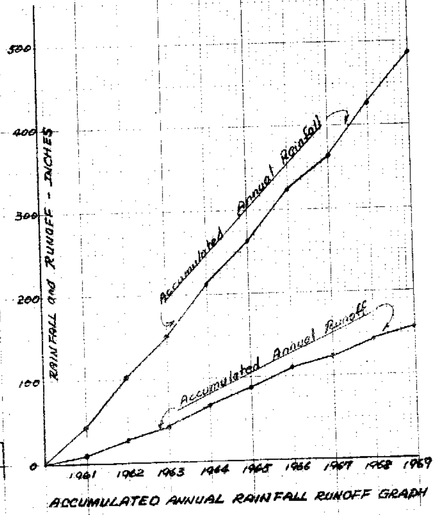


PLATE 10. ANNUAL RAINFALL-RUNOFF RELATIONS

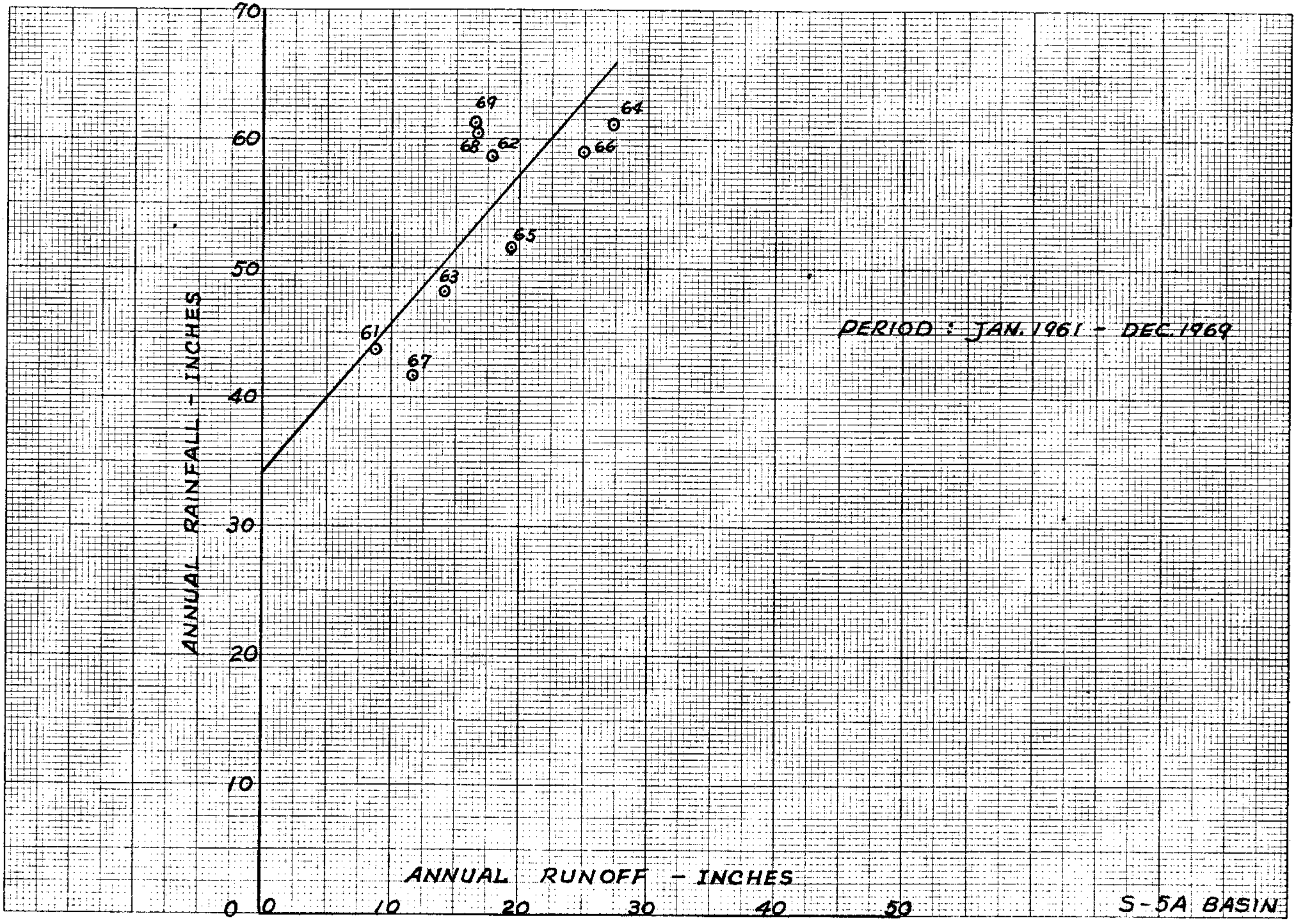


PLATE 11. GENERALIZED RELATION BETWEEN ANNUAL RAINFALL AND RUNOFF

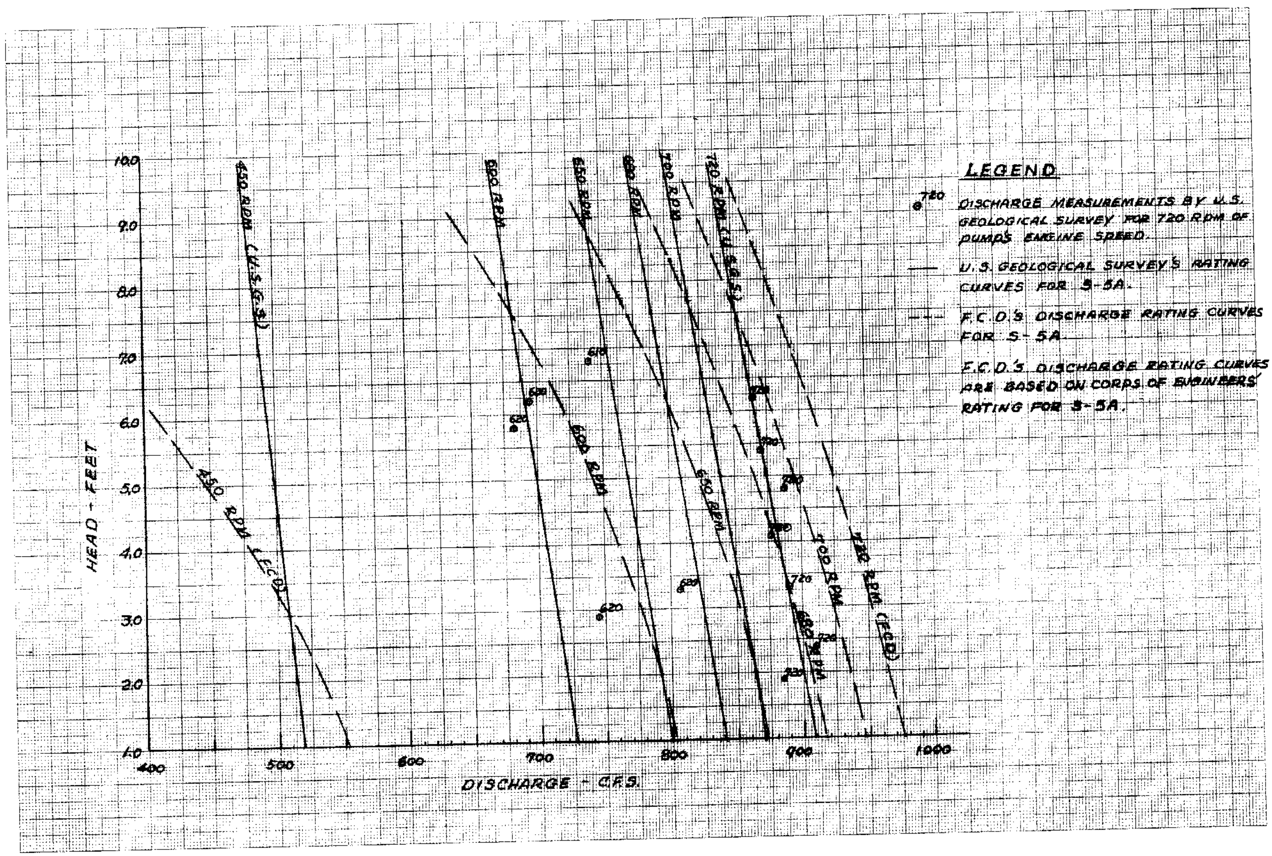


PLATE 12. DISCHARGE RATING FOR S-5A PUMPS

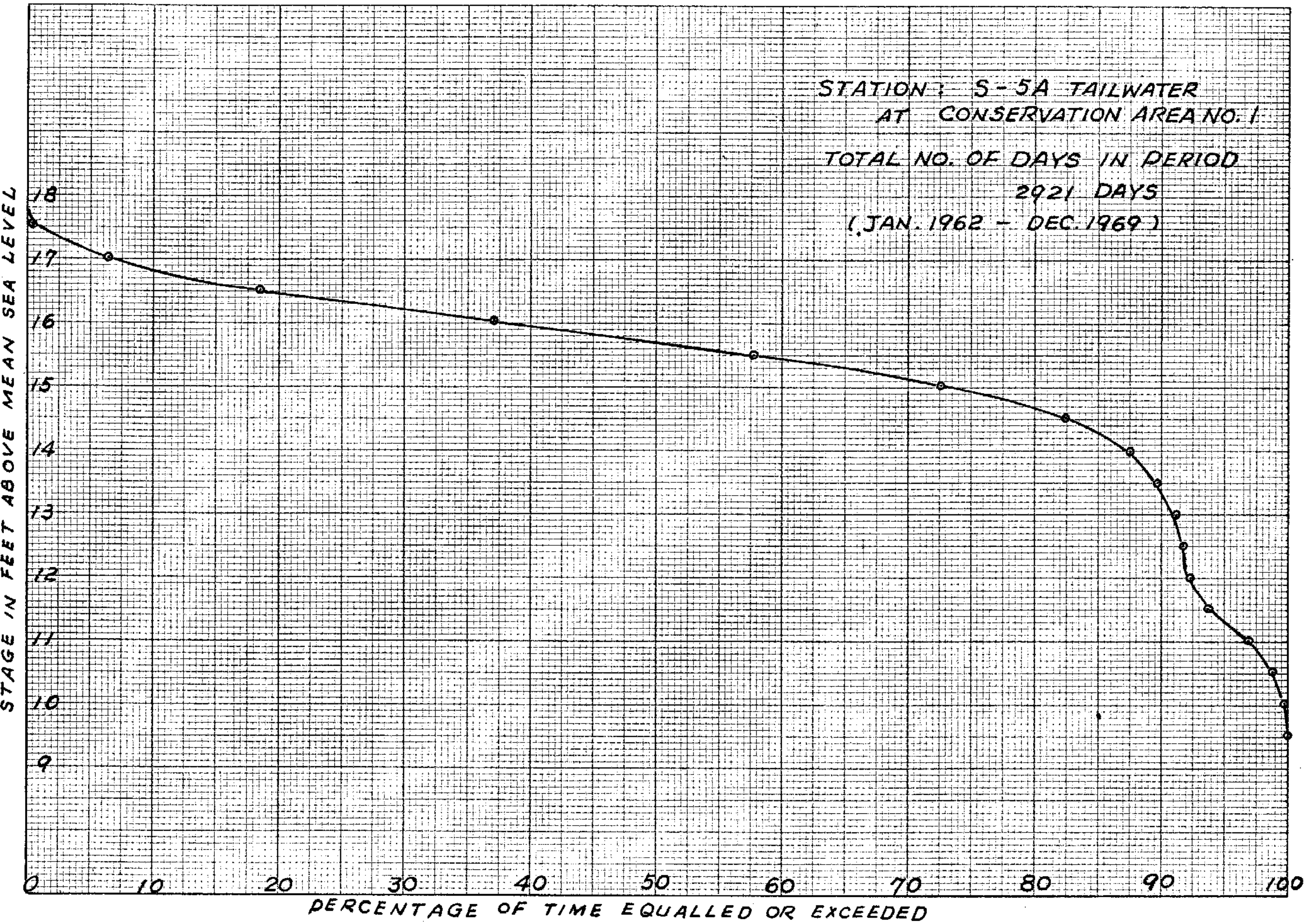


PLATE 13. EXISTING STAGE DURATION OF TAILWATER AT PUMP STATION S-5A

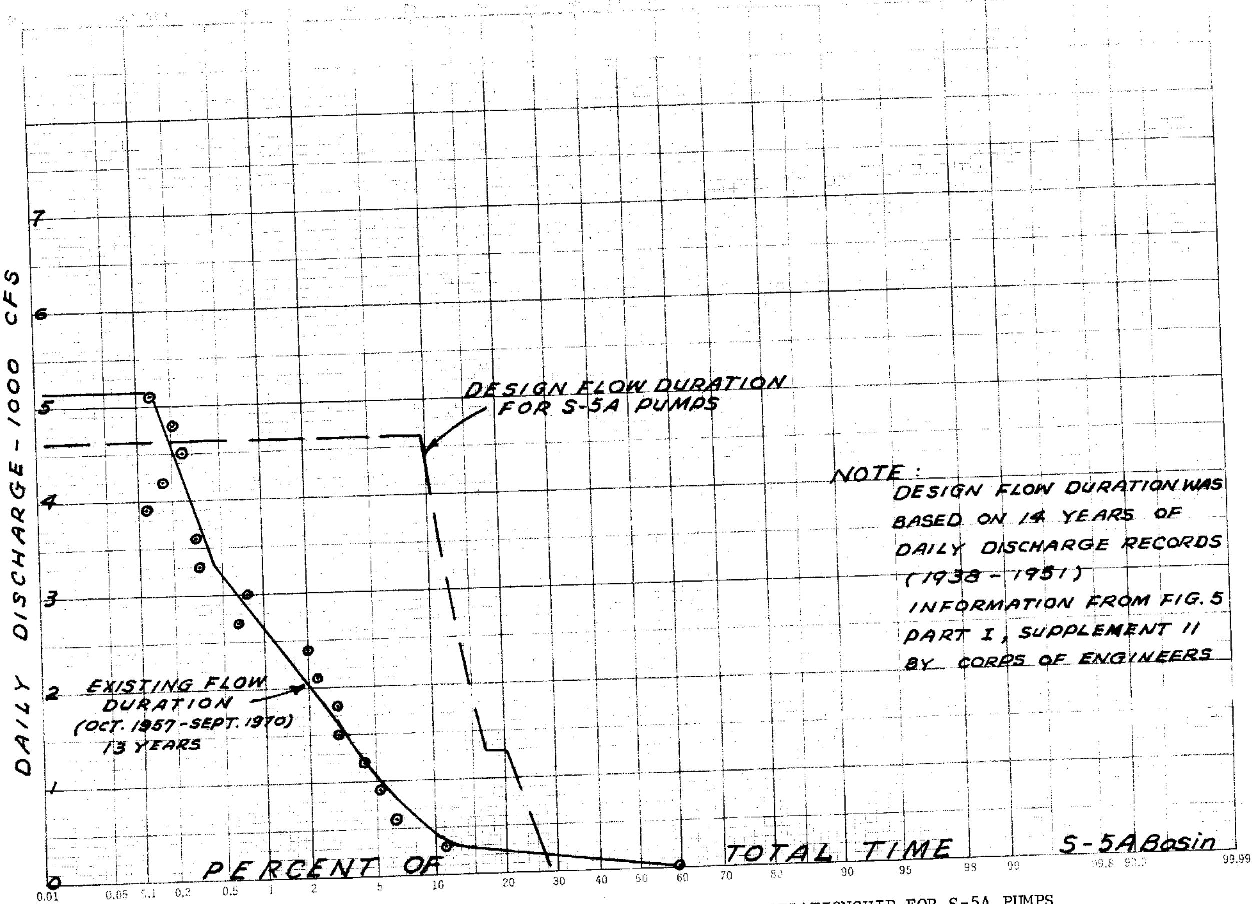


PLATE 14. EXISTING AND DESIGN DISCHARGE DURATION RELATIONSHIP FOR S-5A PUMPS

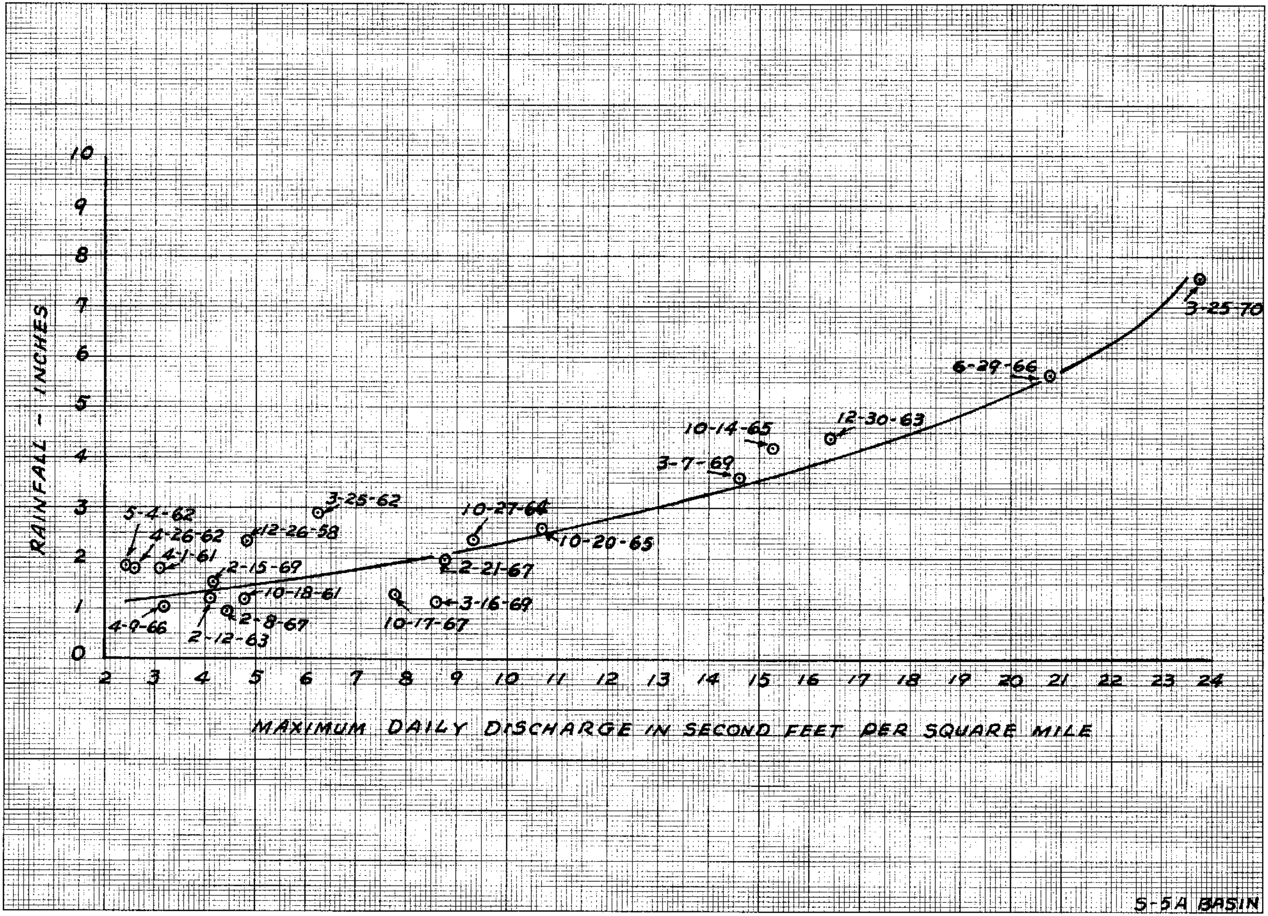


PLATE 15. MAXIMUM DAILY DISCHARGE PER SQUARE MILE IN RELATION TO RAINFALL FOR SELECTED STORM EVENTS

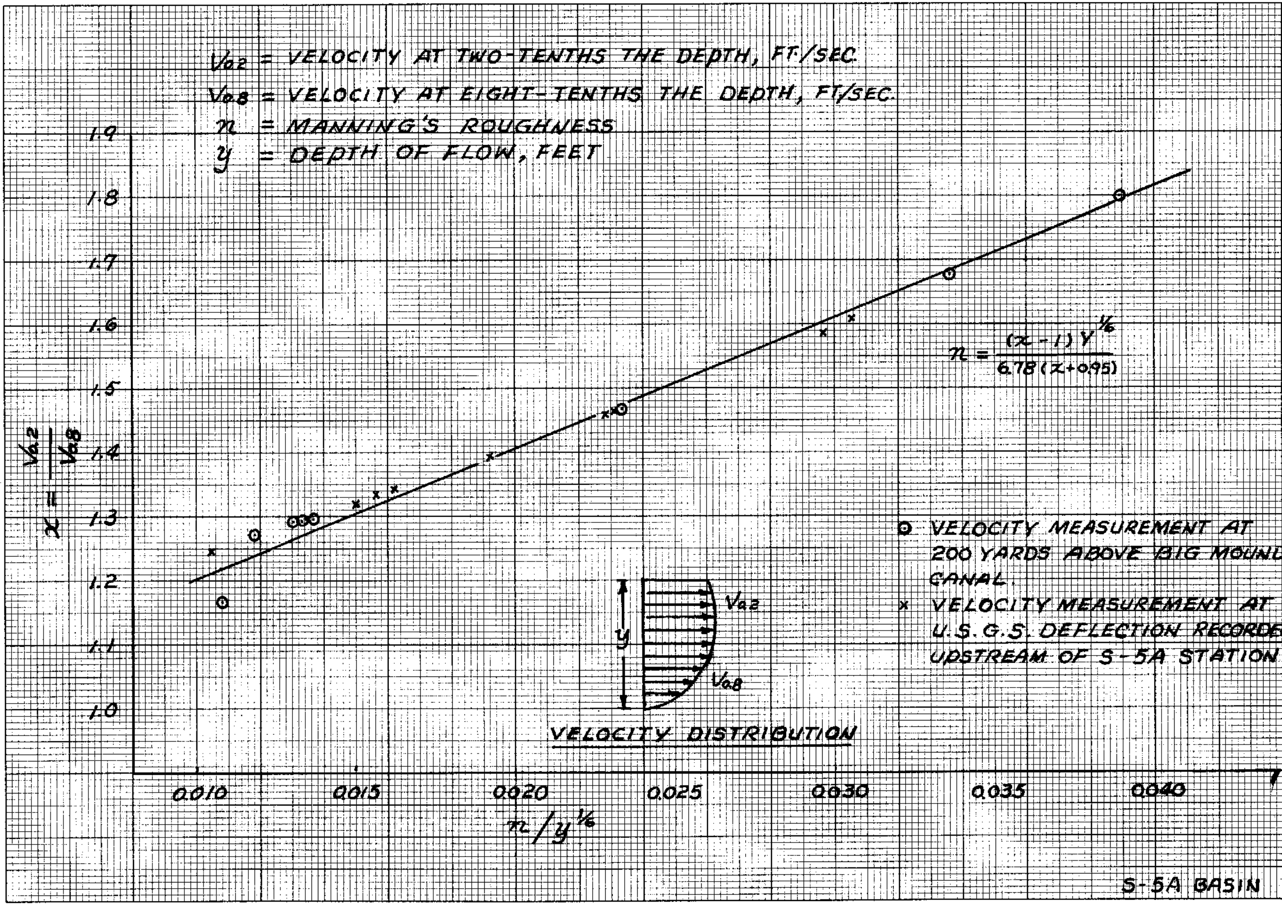


PLATE. 16. RELATION OF MANNING'S "n" AND THE VERTICAL VELOCITY PROFILE BY THE "METHOD OF VELOCITY MEASUREMENT"

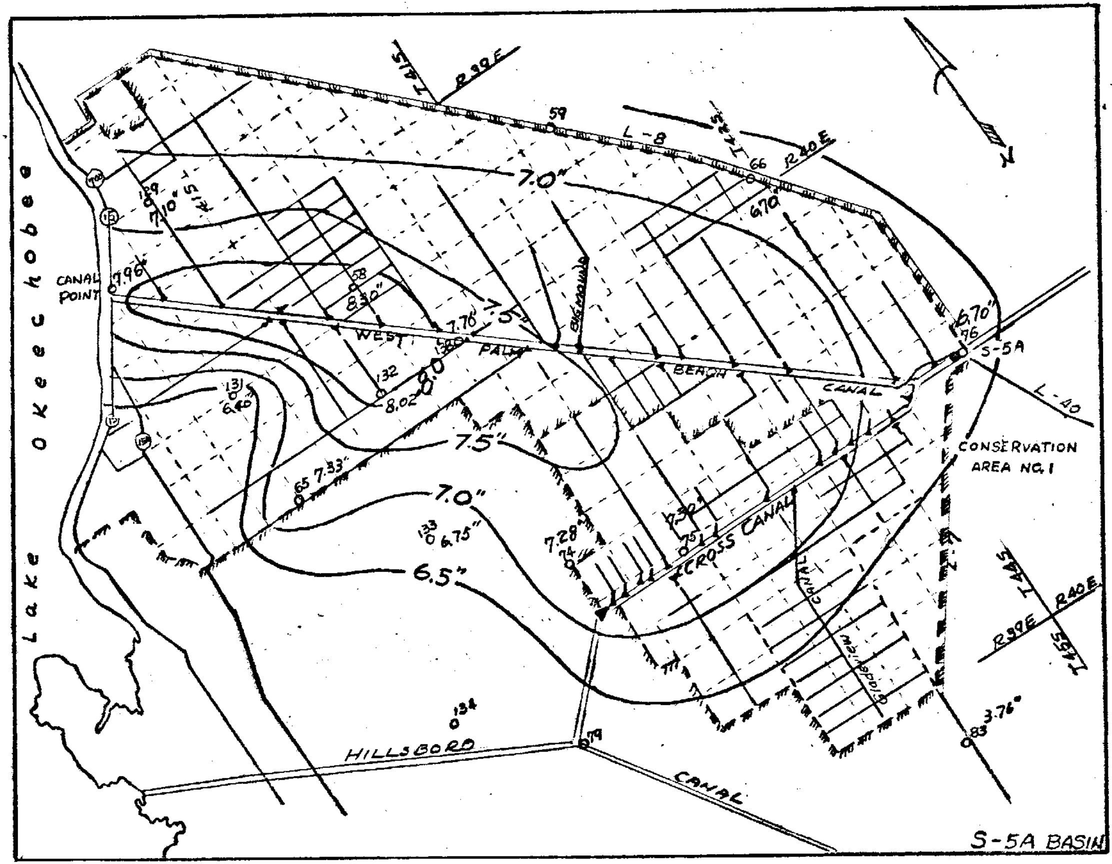


PLATE 18. ISOHYETAL MAP OF THE STORM EVENT OF MARCH 25-26, 1970

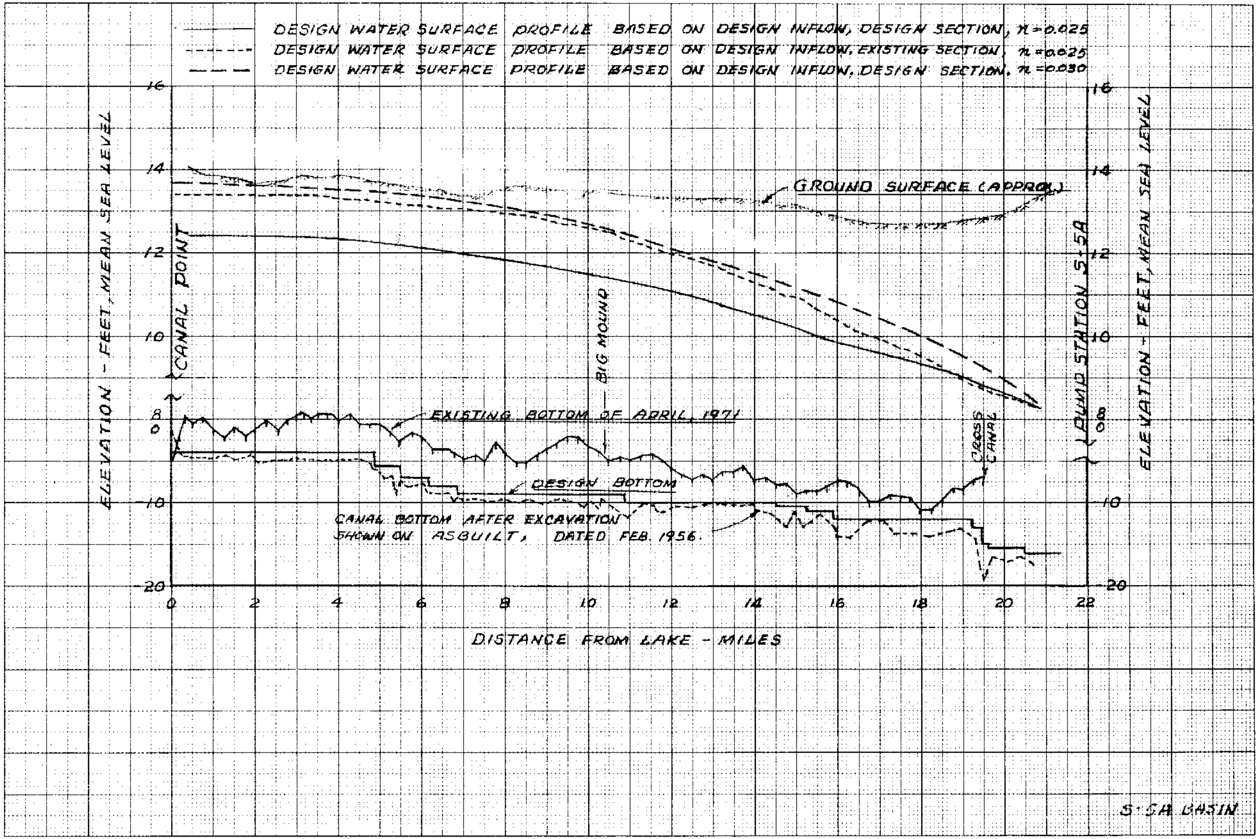


PLATE 17. DESIGN WATER SURFACE PROFILES USING VARIOUS "n" FACTORS PLOTTED WITH EXISTING, DESIGN, AND AS BUILT PROFILES

TABLE I

LAND USE DISTRIBUTION IN S-5A BASIN, 1970

LAND USE	AREA SQ. MILES	PERCENT (%)
Sugar Cane	107.8	48.5
Dairy	45.0	20.3
Vegetables	35.0	15.8
Urban	6.0	2.7
Nature	25.0	11.3
Other	3.0	1.4

NOTE: Other includes citrus and nursery, etc.

Information based on Land Use Information of January 1970.

USCS.

TABLE 2

FARM PUMPS DISCHARGING TO L-10 AND L-12

Pump Index	Area Acres	No. of Unit	Capacity of each Unit GPM	Total Capacity G.P.M.	CFS	in/day	Capacity Allowable under ERF[(a)] CFS	Permit or Application No.
O	3404	2	30,000	60,000	107	0.76	256.0	
AS	54	1	4,000	4,000	9	3.93	26.0	3462
X	2664	2	30,000	60,000	107	0.96	219.0	
AC	5682	4	38,000	152,000	339	1.42	356.5	1901
K1	1280	1	25,000					
		1	10,000	35,000	78	1.45	140.5	
Y1	680	1	30,000	30,000	66.8	2.34	97.2	
B1	200	1	10,000	10,000	22.3	2.65	49.3	3440
Y	5911	2	60,000	120,000	214	0.87	367.0	
AX	6720	4	45,000	180,000	400	1.42	399.0	3277
L2		3	25,000					
L3		1	25,000					
L4		3	25,000					
L1	4160	2	36,000	211,000	470	2.69	291.0	15218
L15		1	30,000					
	700	1	14,000	44,000	98	3.33	99.0	
L5	640	1	18,000	18,000	40	1.49	94.0	15219
L6	640	1	18,000	18,000	40	1.49	94.0	15219
L8	640	1	25,000	25,000	55.7	2.07	94.0	15220
L14	640	1	25,000	25,000	55.7	2.07	94.0	15368
N		3	60,000					
	9485	1	80,000	260,000	579	1.45	504.0	
AA		2	24,000					
L10		1	25,000					13916
L13	2430	2	25,000	123,000	274	2.68	207.4	1035
AM		3	15,000					EDD 10
	1280	1	35,000	80,000	178	3.31	141.0	
AK		1	18,000					
	1600	1	25,000	43,000	95.8	1.43	161.0	3470
F	2885	4	12,000	48,000	85	0.70	230.6	
AB	960	2	25,000	50,000	111.5	2.76	119.0	2953
E	820	2	22,000	44,000	98	2.84	109.0	
AD	900	3	24,000	72,000	160	4.23	114.0	EDD 4
AE	1895	2	36,000	72,000	160.0	2.01	177.8	2740
AF	4220	3	45,000	135,000	300.0	1.70	293.6	3539
AL		2	14,000					
	1235	1	30,000	58,000	129.0	2.49	137.6	2283
AR		1	18,000					
	1600	1	22,000	40,000	71.0	2.14	161.0	EDD 171
A	300	1	15,000	15,000	33.4	2.65	55.0	15021
L7	640	1	18,000	18,000	40.0	1.49	94.0	15219
L9	640	1	25,000	25,000	55.7	2.09	94.0	15230
L12	640	1	25,000	25,000	55.7	2.09	94.0	15232

(a) Everglades Runoff Formula

TABLE 3

FARM PUMPS DISCHARGING TO CROSS CANAL

Pump Index	Area Acres	No. of Unit	Capacity of each Unit GPM	Total Capacity G.P.M.	Total Capacity CFS	Total Capacity in/day	Capacity Allowable under ERS[(a)] CFS	Permit or Application No.
AH	400	2	16,000	32,000	71.3	4.24	72.0	537
A3	2080	1	25,000					
		2	32,000	89,000	198.3	2.27	188.4	189
A4	400	2	35,000	70,000	156.0		72.0	189
AP	5760	4	40,000	160,000	357.0	1.48	315.0	1532
A5	1280	1	35,000	35,000	78.0	1.45	140.6	
AJ	1920	2	31,000	62,000	138.0	1.71	155.0	2634
A7	900	1	18,000	18,000	40.1	1.06	114.4	
A8	1520	1	35,000					
		1	10,000					
		1	40,000	85,000	189.3	2.96	155.6	
A9	700	1	22,000	22,000	49.0	1.67	99.0	
GD1	640	1	7,500					
GD2		1	7,500					15030
GD3		1	25,000	40,000	89.13	3.31	94.0	
GD4	1700	1	7,500					
GD7		3	25,000	82,500	183.8	2.57	166.2	
GD21	960	2	25,000	50,000	111.4	2.76	118.9	15030
GD9	960	1	25,000					
GD10		2	25,000	75,000	167.1	4.14	118.9	15287
GD14	800	2	25,000					
GD16		1	22,000	72,000	160.4	4.77	106.9	15287
GD18	960	2	25,000					
GD20		1	20,000	70,000	156.0	3.87	118.9	15287
GD6	640	1	25,000	25,000	55.7	2.07	94.0	15046
GD5		1	22,000					
		2	25,000					
GD8	1666	2	25,000	122,000	271.8	3.88	164.0	15286
GD11	960			64,000	147.1	3.65	118.9	
GD13	960	1	25,000					
GD15		2	25,000					
GD17		1	6,200	81,200	180.9	4.49	118.9	15047
GD19	960	3	25,000	75,000	167.1	4.14	118.9	
GD22	960	1	25,000	25,000	55.7	1.38	118.9	15288
AU	1280	2	22,500	45,000	100.0	1.86	121.0	EDD 296
AV	760	2	14,000	28,000	62.5	1.96	90.0	EDD 297
AW	320	1	14,000	14,000	31.2	2.32	55.0	EDD 268
A2	760	3	18,000	54,000	120.2	3.76		

(a) Everglades Runoff Formula

TABLE 4

RAINFALL-RUNOFF RELATION AT S-5A BASIN

EVENT Mo-Day-Yr	RAINFALL (1) Acre Ft.	RAINFALL (1) Inches	RUNOFF (2) Acre Ft.	RUNOFF (2) Inches	RATIO (2)/(1)	FREQUENCY in Year
3-25,26-70	88366.94	7.52	70611.00	5.78	0.769	10 *
3-7,8,9-69	41998.00	3.57	21571.34	1.84	0.514	2
3-16-69	12126.23	1.03	10261.53	0.87	0.846	1
2-15-69	17540.79	1.49	4787.94	0.41	0.273	1
10-17-67	14266.81	1.21	5849.00	0.50	0.410	1
2-8,9-67	10969.28	0.93	4125.00	0.35	0.376	1
2-21-67	22310.95	1.90	6322.00	0.54	0.283	1
4-9,10-66	11589.73	0.98	1836.00	0.16	0.158	1
6-29,30-66	65918.79	5.61	43365.00	3.69	0.658	4 - 5 *
10-14,15-65	48857.47	4.16	21928.00	1.87	0.449	2 - 3 *
10-20,21,22-65	29837.15	2.54	27062.00	2.30	0.907	1
10-27,28-64	27516.82	2.34	20926.00	1.78	0.760	1
2-12-63	13847.45	1.18]	2754.00	0.23	0.199	1
12-30,31-63	51130.73	4.35	35073.00	2.98	0.686	3
4-26-62	20900.94	1.78	2080.00	0.18	0.099	1
5-4,5,6-62	22109.16	1.88	7368.00	0.63	0.333	1
3-25,26-62	33782.12	2.88	6308.00	0.54	0.187	1
10-18-61	13642.32	1.16	2542.00	0.22	0.186	1
4-1-61	20380.43	1.73	2454.00	0.21	0.120	1
12-26,27-58	26994.52	2.30	7227.00	0.62	0.268	1

*Approximate Figure

TABLE 5

RUNOFF EVALUATION FOR THE EVENT OF JUNE 29-30, 1966 AND MARCH 25-26, 1970

STORM EVENT: June 29-30, 1966

DATE	S-5A PUMP AC.FT.	S-5A WEST AC.FT.	CANAL POINT AC.FT.	NET DISCH. AC.FT.	DEPTH INCHES/DAY
6-29-66	6327	-1120	0	5207	0.44
6-30-66	9600	-1080	0	8520	0.73
7-1-66	9858	-754	0	9104	0.78
7-2-66	9859	-2480	0	7378	0.63
7-3-66	8351	-2470	0	5881	0.50
7-4-66	5118	-1890	0	3228	0.27
7-5-66	2638	-940	0	1698	0.14
7-6-66	1595	-482	0	1113	0.10
7-7-66	1696	-460	0	1236	0.10
TOTAL				43365	3.69

Runoff terminated on July 7, 1966

STORM EVENT: March 25-26, 1970

DATE	S-5A PUMP AC.FT.	S-5A WEST AC.FT.	CANAL POINT AC.FT.	NET DISCH. AC. FT.	DEPTH INCHES/DAY
3-25-70	1274	96	0	1370	0.117
3-26-70	9992	-365	0	9627	0.819
3-27-70	10991	0	0	10991	0.945
3-28-70	10818	0	0	10818	0.921
3-29-70	10160	-312	0	9848	0.838
3-30-70	9832	-1745	0	9814	0.836
3-31-70	8057	-2800	0	5257	0.447
4-1-70	6009	-2940	0	3069	0.261
4-2-70	4059	-1800	0	2258	0.192
4-3-70	3110	-1540	0	1570	0.133
4-4-70	2740	-1670	0	1070	0.091
4-5-70	4281	-2090	0	2190	0.186
TOTAL				70611	5.780

Runoff terminated on April 5, 1970

TABLE 6

COMPARISON OF THE EVERGLADES RUNOFF FORMULA WITH NEW CURVE

Drainage Area Sq.Mi.	New Everglades Runoff Curve: Runoff cfs.per sq. mi.	Discharge cfs.	Capacity in/day	Existing Everglades Runoff Curve: Runoff cfs.per sq. mi.	Discharge cfs.	Capacity in/day
0.5	70.85	35.4	2.64	127.6	63.8	4.75
1.0	55.00	55.0	2.05	94.0	94.0	3.50
4.0	36.00	144.0	1.34	53.5	214.0	1.99
140.0	20.03	2804.0	0.74	19.9	2786.0	0.74
221.0	19.37	4280.0	0.72	18.4	4066.0	0.68

TABLE 7

COMPARISON OF RUNOFF FROM SUGARCANE, IMPROVED PASTURE, AND TRUCK FARM ACREAGES FOR 1 AND 2 SQUARE MILE AREAS

LAND USE	Drainage Area = 1 Sq. Mile: Pump Capacity Cfs.	In./day	No. of days required to remove 3.4" *	Drainage Area = 2 Sq. Miles: Pump Capacity cfs.	In./day	No. of days required to remove 3.4" *
Truck Farm	63	2.34	1.45	160	2.97	1.14
Improved Pasture	58	2.16	1.57	100	1.86	1.83
Sugarcane	55	2.04	1.67	98	1.82	1.87

Note: * 3.4" is the design rainfall excess in the basin.

<u>TABLE 8</u>

DERIVATION OF SYNTHETIC INFLOW AND DISCHARGE HYDROGRAPHS IF THE EXISTING PUMPS ARE LIMITED TO 3/4 INCH PER DAY.
LOCATION: West Palm Beach Canal above junction of Cross Canal
CONDITION: March, 1970 Event

DURATION DAYS	INFLOW: CFS/DAY PUMP	GRAVITY	TOTAL	ACCUMULATED INFLOW CFS/DAY	ACCUMULATED DISCHARGE CFS/DAY	DISCHARGE CFS
1	420	0	420	420	220	220
2	2060	300	2360	2780	2350	2070
3	2060	300	2360	5140	4800	2450
4	2060	300	2360	7500	7250	2450
5	2060	300	2360	9860	9540	2290
6	2060	300	2360	12220	11960	2420
7	2060	300	2360	14580	14800	2840
8	2060	300	2360	16940	16830	2030
9	1030	650	1680	18620	18350	1520
10	0	1400	1400	20020	19850	1500
11	0	1200	1200	21220	21150	1300
12	0	1000	1000	22220	22150	1000
13	0	900	900	23120	23120	900
14	0	900	900	24020	24020	900
15	0	700	700	24720	24720	700
16	0	600	600	25320	25320	600
17	0	500	500	25820	25820	500
18	0	300	300	26120	26120	300

TABLE 9

DERIVATION OF SYNTHETIC INFLOW AND DISCHARGE HYDROGRAPHS IF THE EXISTING PUMPS ARE LIMITED TO 3/4 INCH PER DAY.
LOCATION: Cross Canal at Discharge to West Palm Beach Canal
CONDITION: March, 1970 Event

DAY	3/4 INCH INFLOW CFS RATE CFS/DAY	3/4 INCH INFLOW CFS ACCUMULATED CFS/DAY	SYNTHETIC DISCHARGE ACCUMULATED CFS/DAY	SYNTHETIC DISCHARGE RATE CFS/DAY
1	194	194	5	5
2	970	1164	300	250
3	970	2134	700	400
4	970	3104	1020	420
5	970	4074	1480	460
6	970	5044	2160	680
7	970	6014	2850	690
8	970	6984	4180	1230
9	485	7469	5310	1130
10	0	7469	5840	530
11	0	7469	6260	420
12	0	7469	6440	180
13	0	7469	6520	80
14	0	7469	6570	50
15	0	7469	6795	225
16	0	7469	7245	450
17	0	7469	7470	225

TABLE 10

DERIVATION OF INFLOW AND DISCHARGE HYDROGRAPHS FOR ACTUAL 1970 DEVELOPMENT
LOCATION: West Palm Beach Canal above junction of Cross Canal
CONDITION: March, 1970 Event

DURATION DAYS	INFLOW CFS/DAY	ACCUMULATED INFLOW CFS/DAY	DISCHARGE CFS/DAY	ACCUMULATED DISCHARGE CFS/DAY
1	717	717	558	558
2	3317	4034	3290	3848
3	3774	7808	3787	7635
4	3720	11528	3767	11402
5	3414	14932	3434	14836
6	2882	17814	2898	17734
7	1953	19767	1958	19692
8	1486	21253	1494	21186
9	1120	22373	1123	22309
10	888	23261	891	23200
11	840	24101	855	24055
12	943	25044	958	25013
13	630	25674	638	25651
14	446	26120	469	26120

TABLE 11

DERIVATION OF INFLOW AND DISCHARGE HYDROGRAPHS FOR ACTUAL 1970 DEVELOPMENT
LOCATION: Cross Canal at Discharge to West Palm Beach Canal
CONDITION: March, 1970 Event

DAY	PUMP INFLOW EXISTING CFS/DAY	PUMP INFLOW ACCUMULATED CFS/DAY	DISCHARGE FROM COMPUTED HYDRO. CFS/DAY	DISCHARGE FROM COMPUTED HYDRO. ACCUMULATED CFS/DAY
1	669.0	669.0	117.0	117.0
2	3244.0	3913.0	1240.0	1357.0
3	1977.0	5890.0	1330.0	2687.0
4	968.0	6858.0	1275.0	3962.0
5	486.0	7344.0	1144.0	5106.0
6	40.0	7384.0	533.0	5639.0
7	0	7384.0	417.0	6056.0
8	0	7384.0	189.0	6245.0
9	0	7384.0	87.0	6332.0
10	0	7384.0	56.0	6388.0
11	0	7384.0	273.0	6661.0
12	0	7384.0	583.0	7244.0
13	0	7384.0	250.0	7494.0
14	0	7384.0	0	7494.0

TABLE 12

DERIVATION OF INFLOW AND DISCHARGE HYDROGRAPHS FOR TOTAL DEVELOPMENT.
CONDITION: March, 1970 Event

DURATION DAYS	COL. 1 cfs	COL. 2 cfs	COL. 3 cfs	COL. 4 cfs	COL. 5 cfs	COL. 6 cfs	COL. 7 cfs	COL. 8 cfs	COL. 9 cfs
1	368	717	349	787	1136	1504	1504	1300	1304
2	1258	3317	2059	1552	3611	4869	6373	6100	4800
3	1330	3774	2444	1377	3377	4707	11080	10700	4600
4	1424	3720	2296	957	3253	4677	15757	15300	4600
5	1283	3414	2131	270	2401	3684	19441	19300	4000
6	985	2882	1897	55	1951	2937	22378	22300	3000
7	607	1953	1346	0	1346	1953	24331	24300	2000
8	471	1486	1015	0	0	641	24972	24972	641
9	416	1120	704	0	0	416	25388	25388	416
10	316	888	572	0	0	316	25704	25704	316
11	300	840	540	0	0	300	26004	26004	300
12	472	943	471	0	0	472	26476	26472	472
13	424	630	206	0	0	424	26900	26900	424
14	346	446	100	0	0	340	27240	27240	346

Note:

Column 1: Inflow from Reach 1, i.e., the discharge computed from Reach 1 shown on Plate 23.

Column 2: Computed inflow at location above the junction of Cross Canal and West Palm Beach Canal shown on Table 10.

Column 3: Difference between Columns 2 and 1.

Column 4: Additional pump inflow from future developed areas.

Column 5: Sum of Columns 3 and 4.

Column 6: Sum of Columns 5 and 1 (i.e., estimated future inflow).

Column 7: Summation of daily value shown in Column 6.

Column 8: Using Column 7 and the relation of the mass inflow and outflow shown on Table 10 to estimate the mass discharge for future condition.

Column 9: Estimated future discharge.

TABLE 13

COMPARISON OF EVERGLADES AGRICULTURAL INFLOW CURVE WITH EVERGLADES RUNOFF FORMULA

	EVERGLADES AGRICULTURAL INFLOW CURVE			EVERGLADES RUNOFF FORMULA		
DRAINAGE AREA SQ.MILES	Runoff cfs per sq. mi.	Discharge cfs	Capacity in/day	Runoff cfs per sq. mi.	Discharge cfs	Capacity in/day
0.5	69.5	34.7	2.57	127.6	63.8	4.75
1.0	60.5	60.5	2.23	94.0	94.0	3.50
4.0	45.8	183.3	1.70	53.5	214.0	1.99
140.0	22.5	3150.0	0.84	19.9	2786.0	0.74
221.0	20.5	4530.0	0.76	18.4	4066.0	0.68

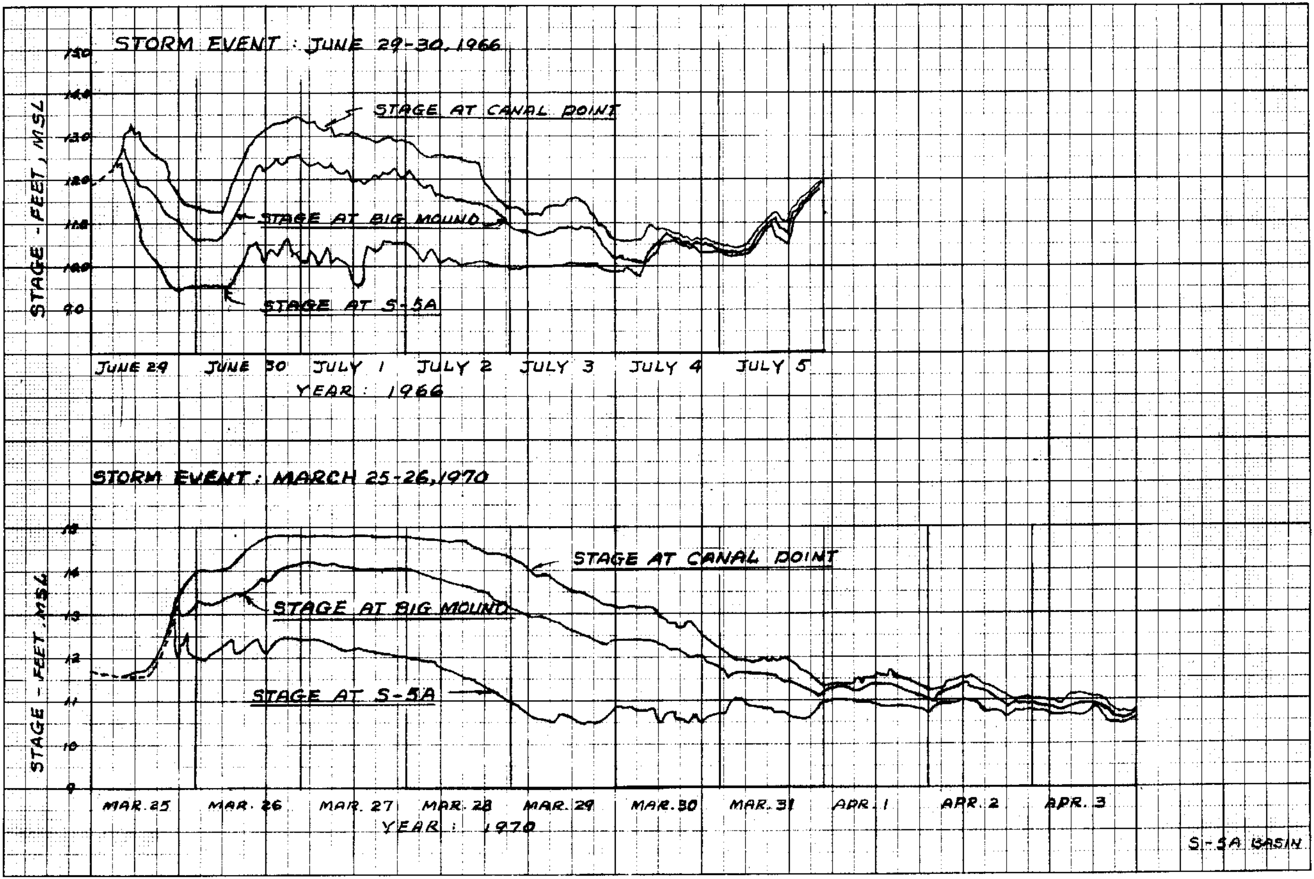


PLATE 19. STAGE HYDROGRAPHS AT CANAL POINT, BIG MOUND, AND S-5A FOR STORM EVENTS OF JUNE, 1966, AND MARCH, 1970

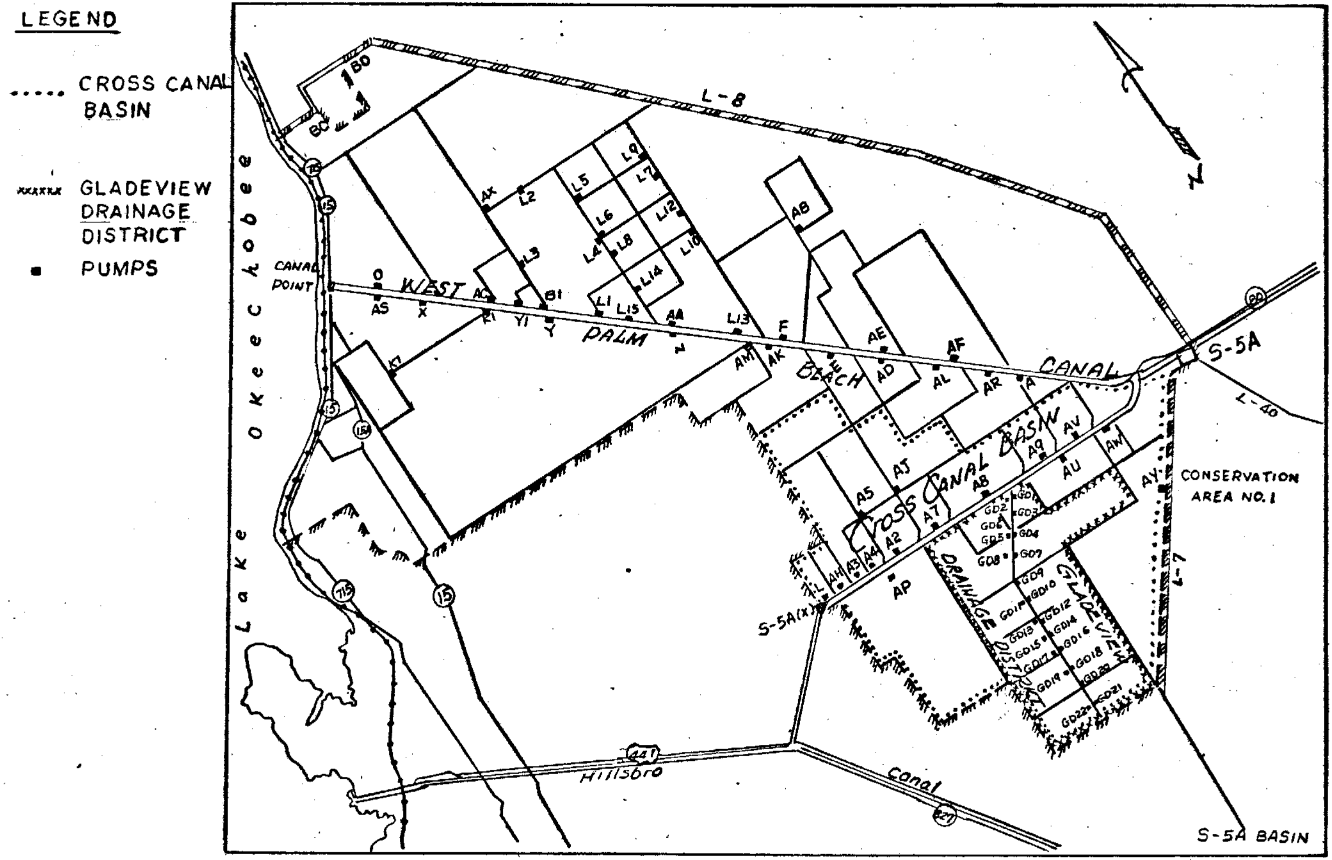


PLATE 20. LOCATIONS OF THE EXISTING FARM PUMPS AND AREA SERVED

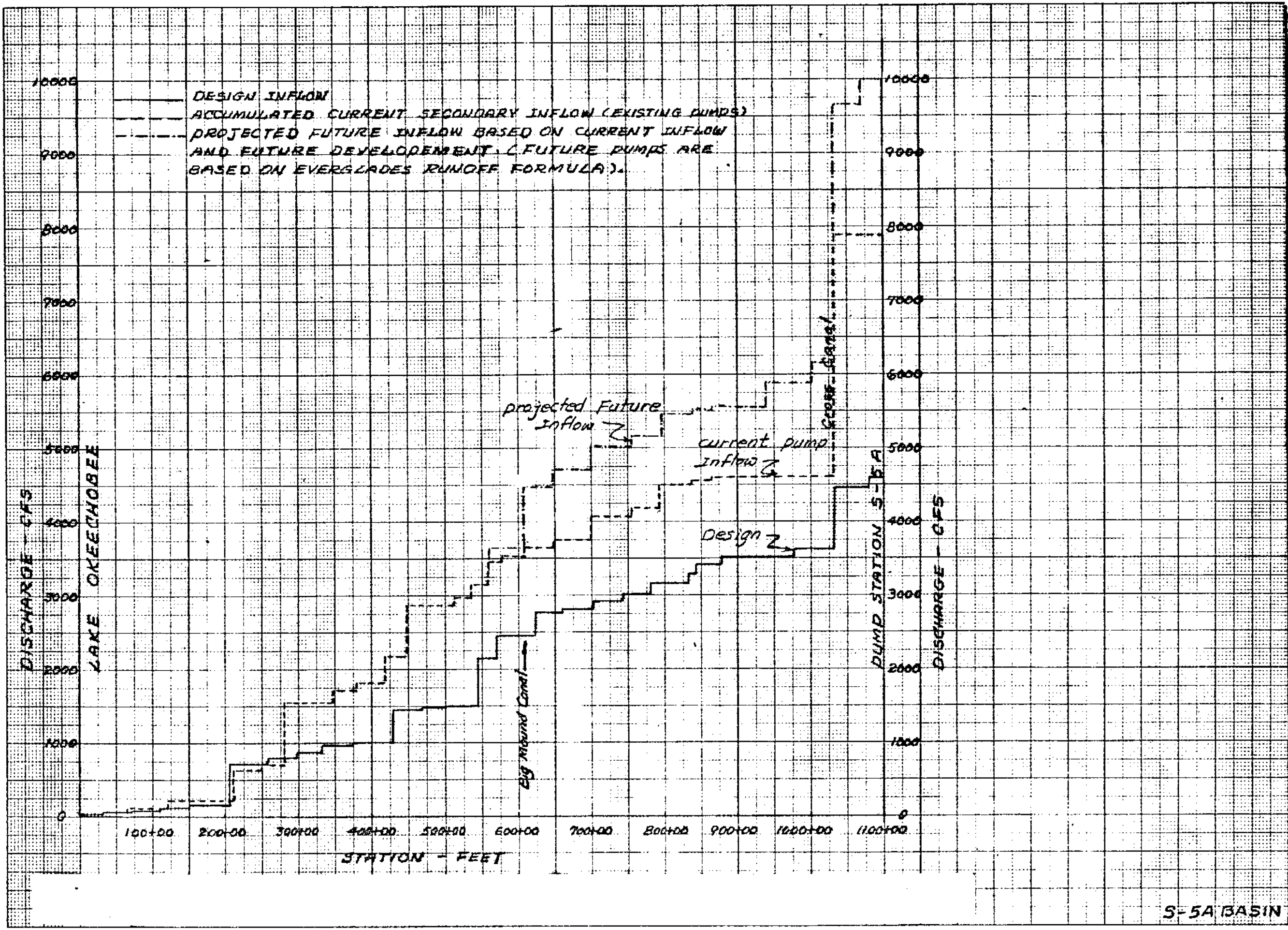


PLATE 21. PRESENT, PROJECTED, AND DESIGN INFLOWS IN THE WEST PALM BEACH CANAL

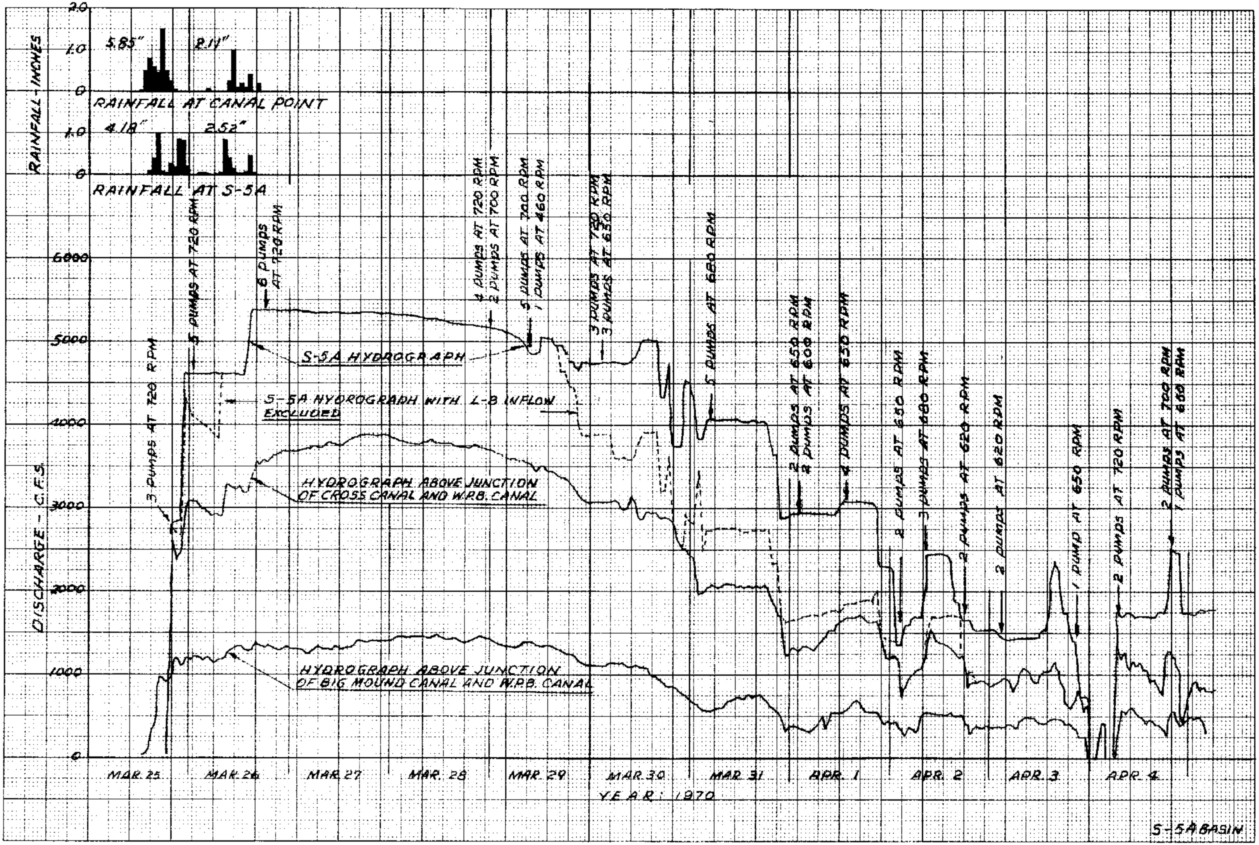
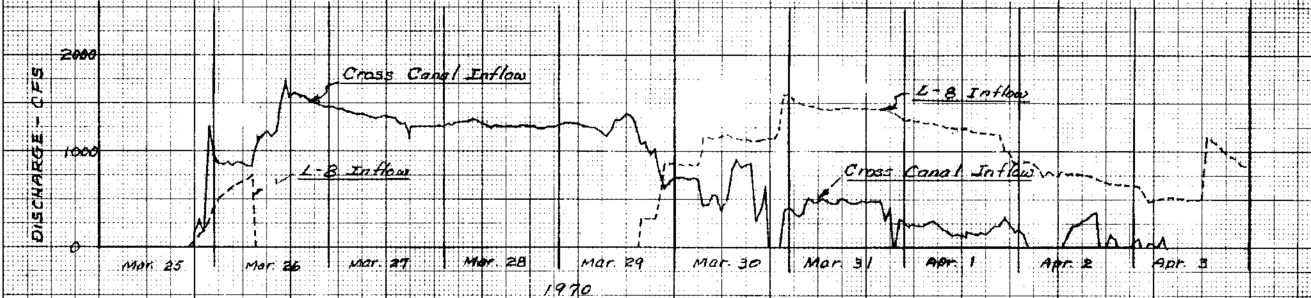
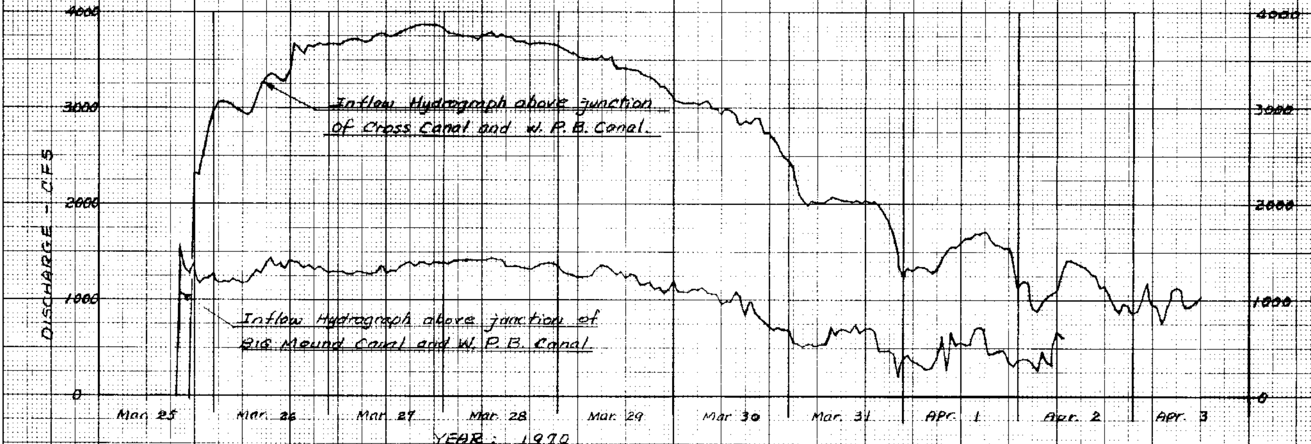


PLATE 22. DISCHARGE HYDROGRAPHS AT THE THREE PRIMARY LOCATIONS FOR STORM EVENT OF MARCH 25-26, 1970



Synthetic Secondary Inflow Hydrographs AT outlet of Cross Canal for Event of March 25, 26, 1970



Synthetic Secondary Inflow Hydrographs for Storm Event of March 25, 26, 1970

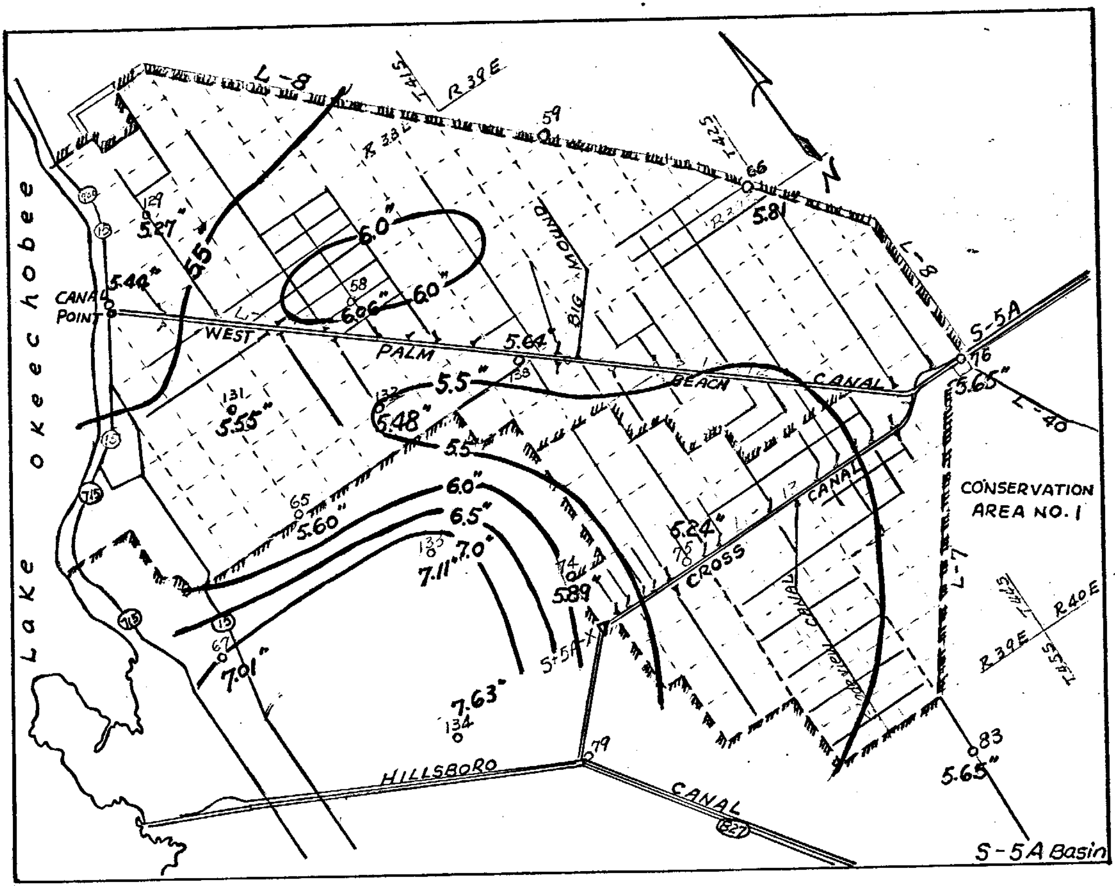


PLATE 24. ISOHYETAL MAP FOR STORM EVENT OF JUNE 29-30, 1966

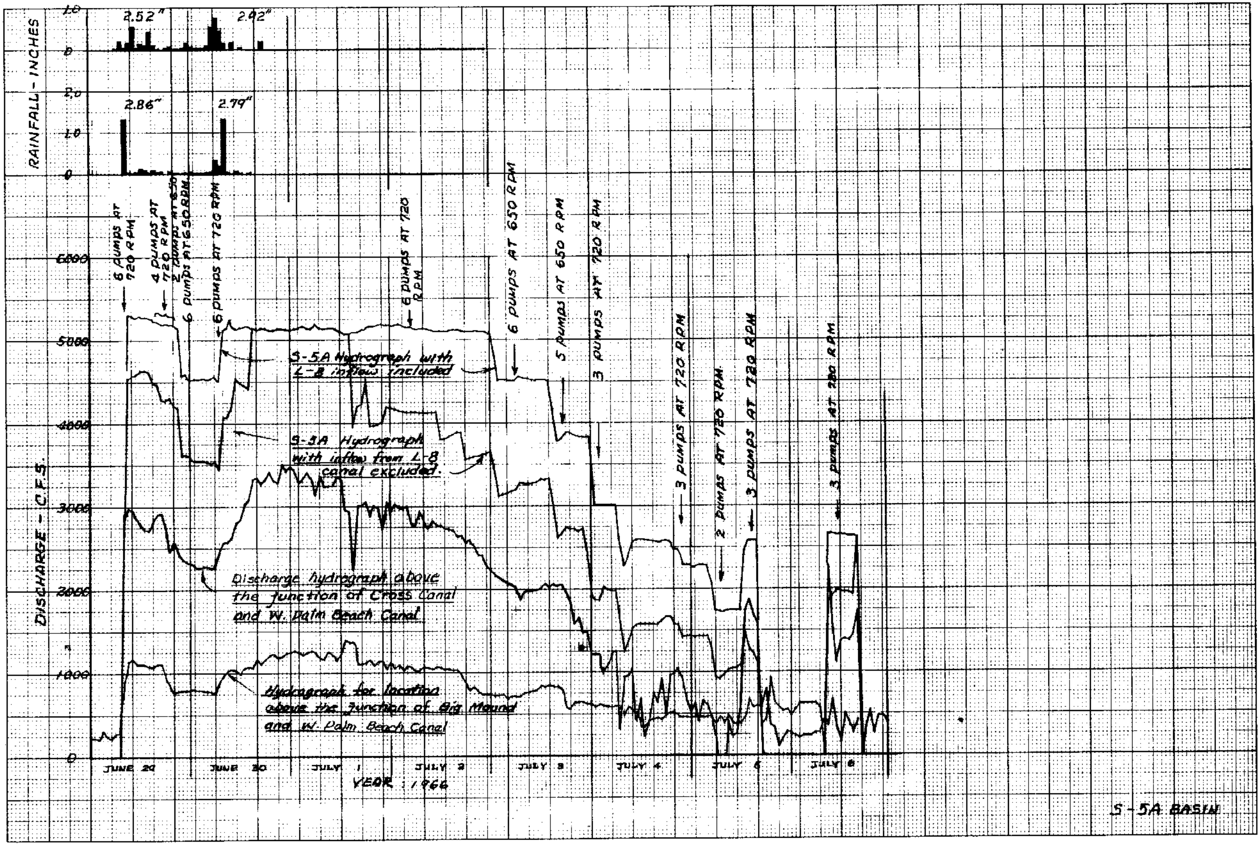


PLATE 25. DISCHARGE HYDROGRAPHS AT THREE PRIMARY LOCATIONS FOR THE STORM EVENT OF JUNE 29-30, 1966

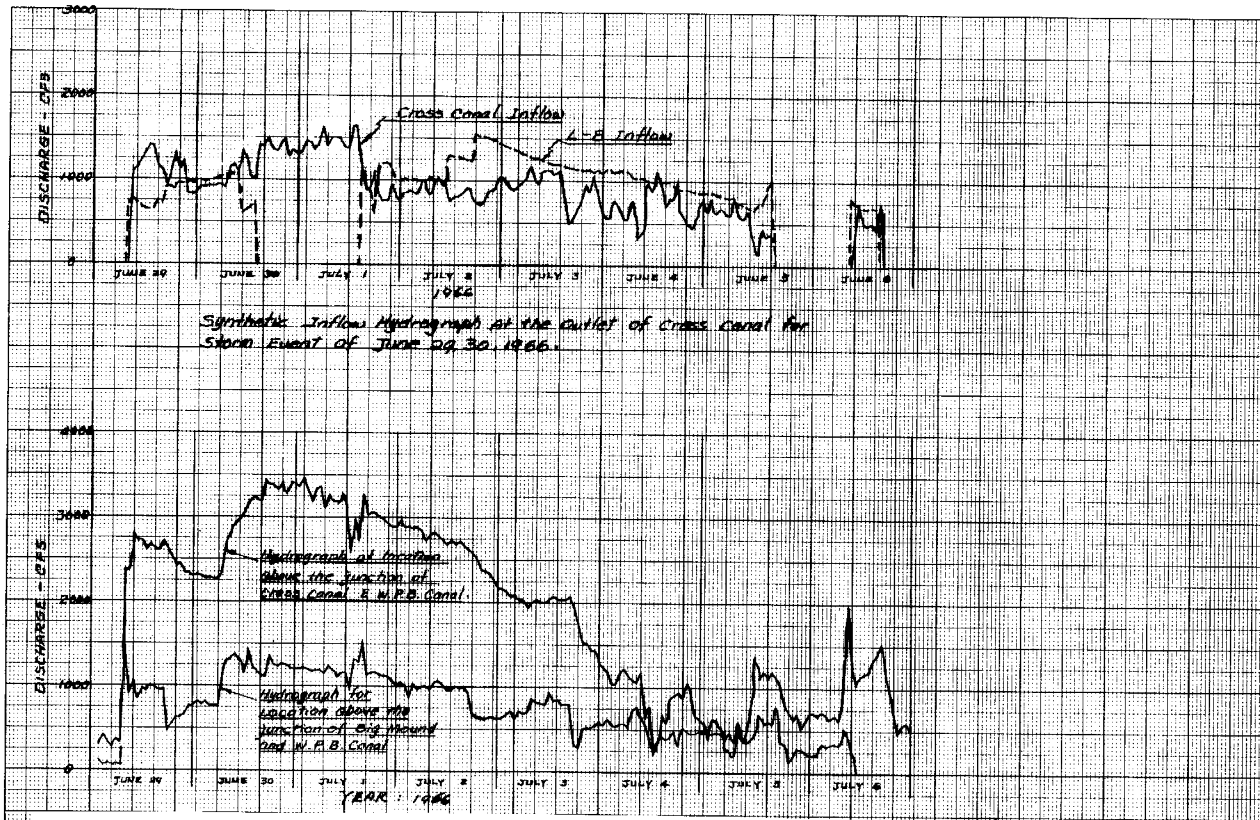


PLATE 26. DERIVED INFLOW HYDROGRAPHS AT THREE PRIMARY LOCATIONS FOR THE STORM EVENT OF JUNE 29-30, 1966.

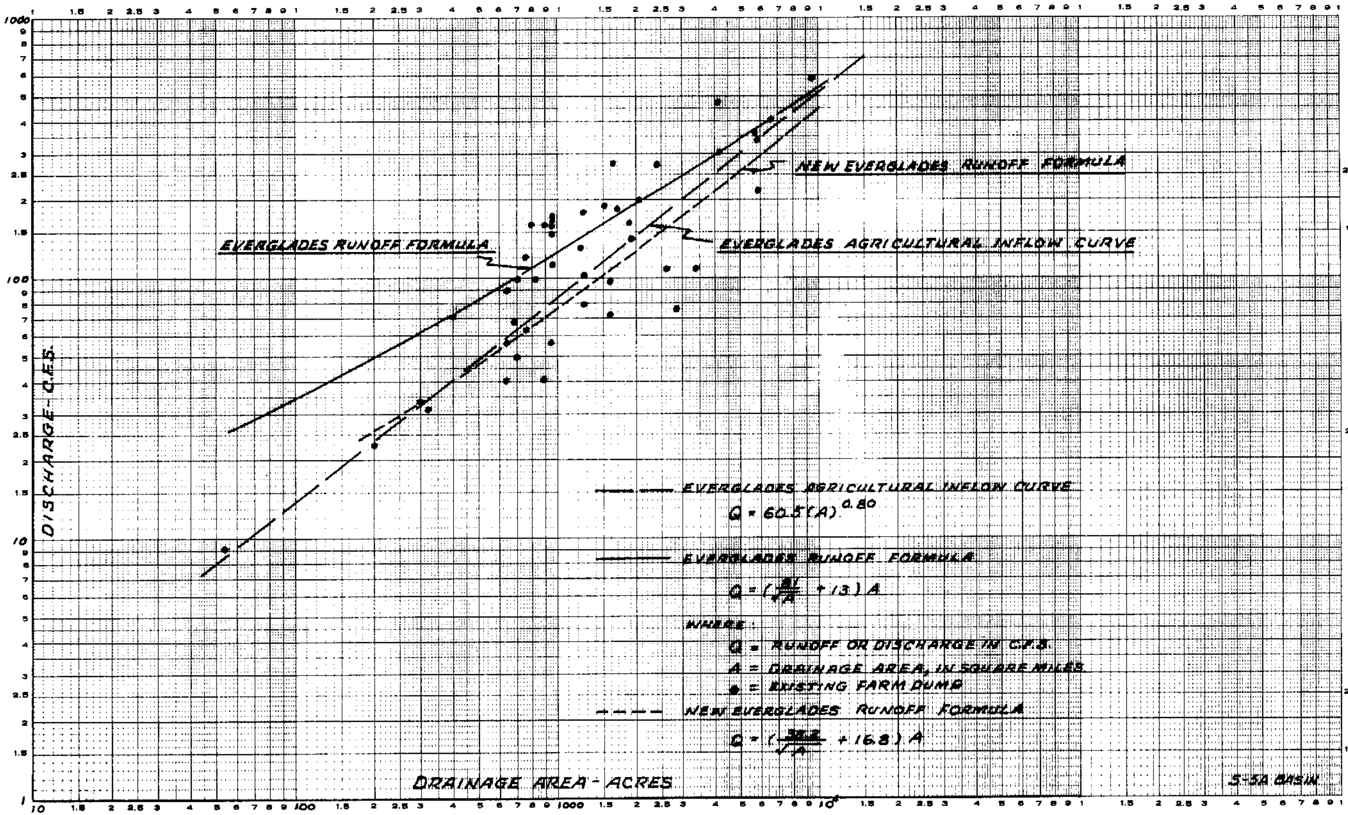


PLATE 27. COMPARISON OF EXISTING FARM PUMPAGE, EVERGLADES RUNOFF FORMULA, AND ALTERNATIVE RUNOFF CURVES

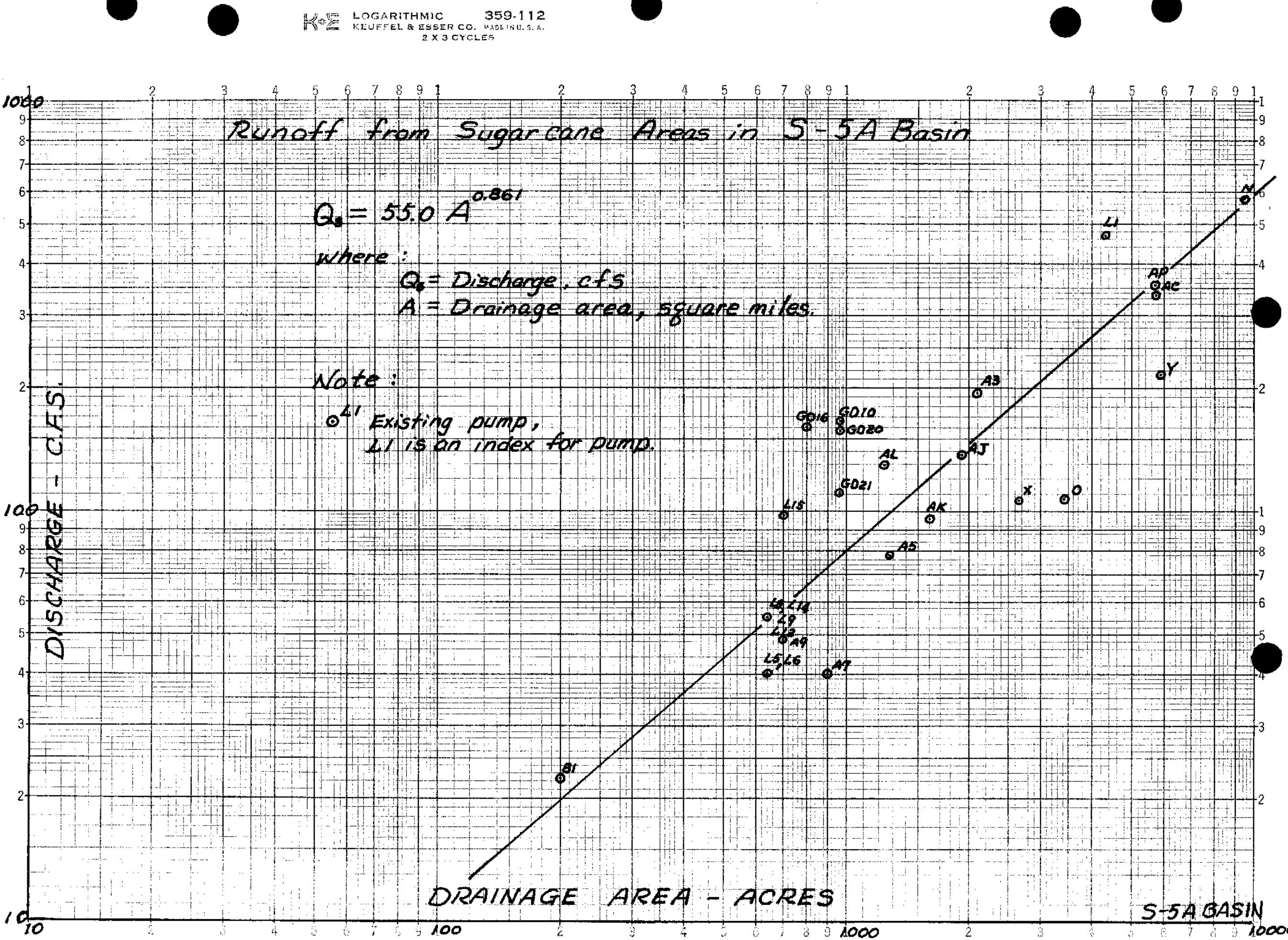


PLATE 28. DISCHARGE-AREA RELATION OF SUGARCANE AREA PUMPAGE

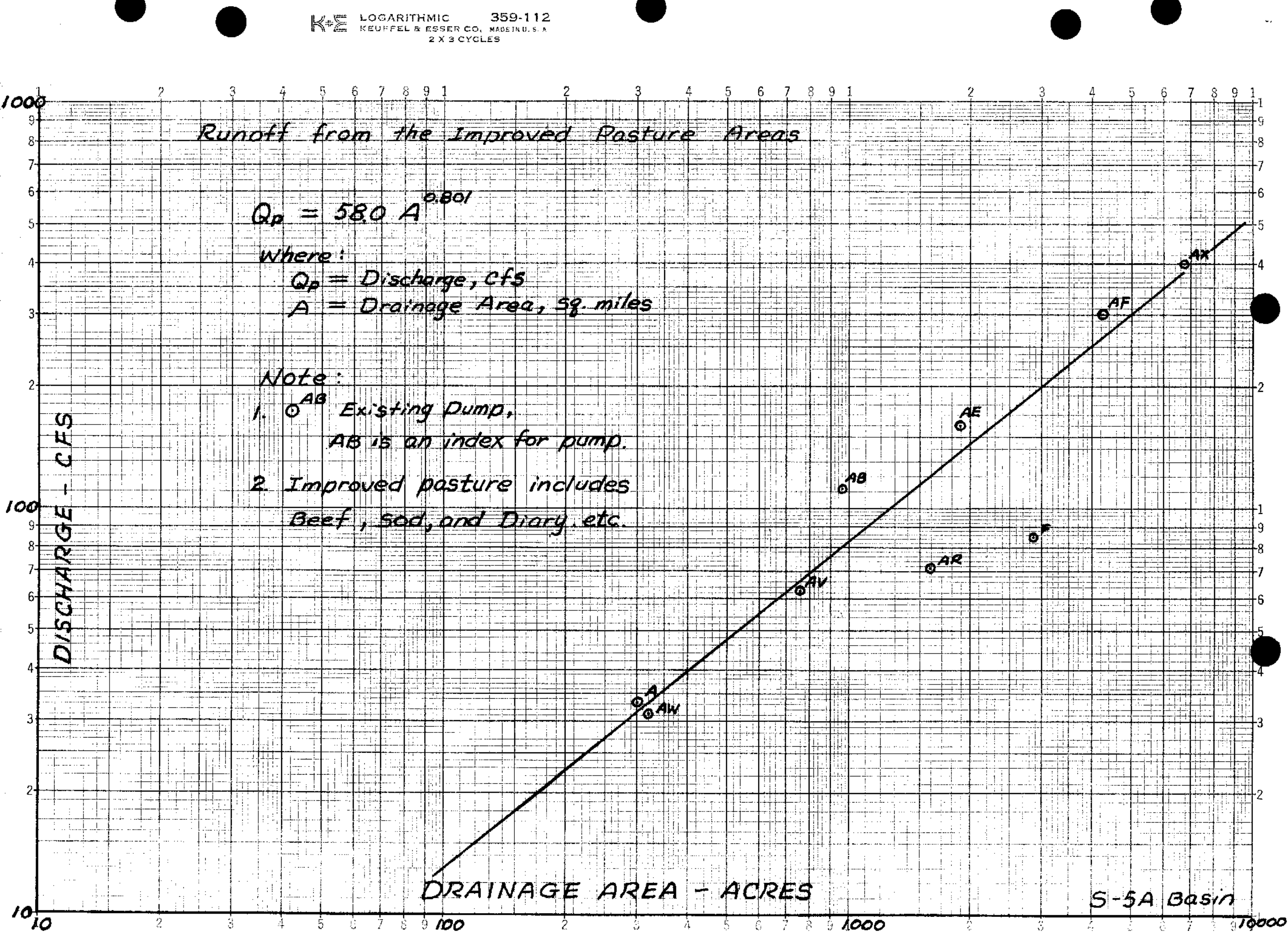


PLATE 29. DISCHARGE-AREA RELATION OF IMPROVED PASTURE AREA PUMPAGE

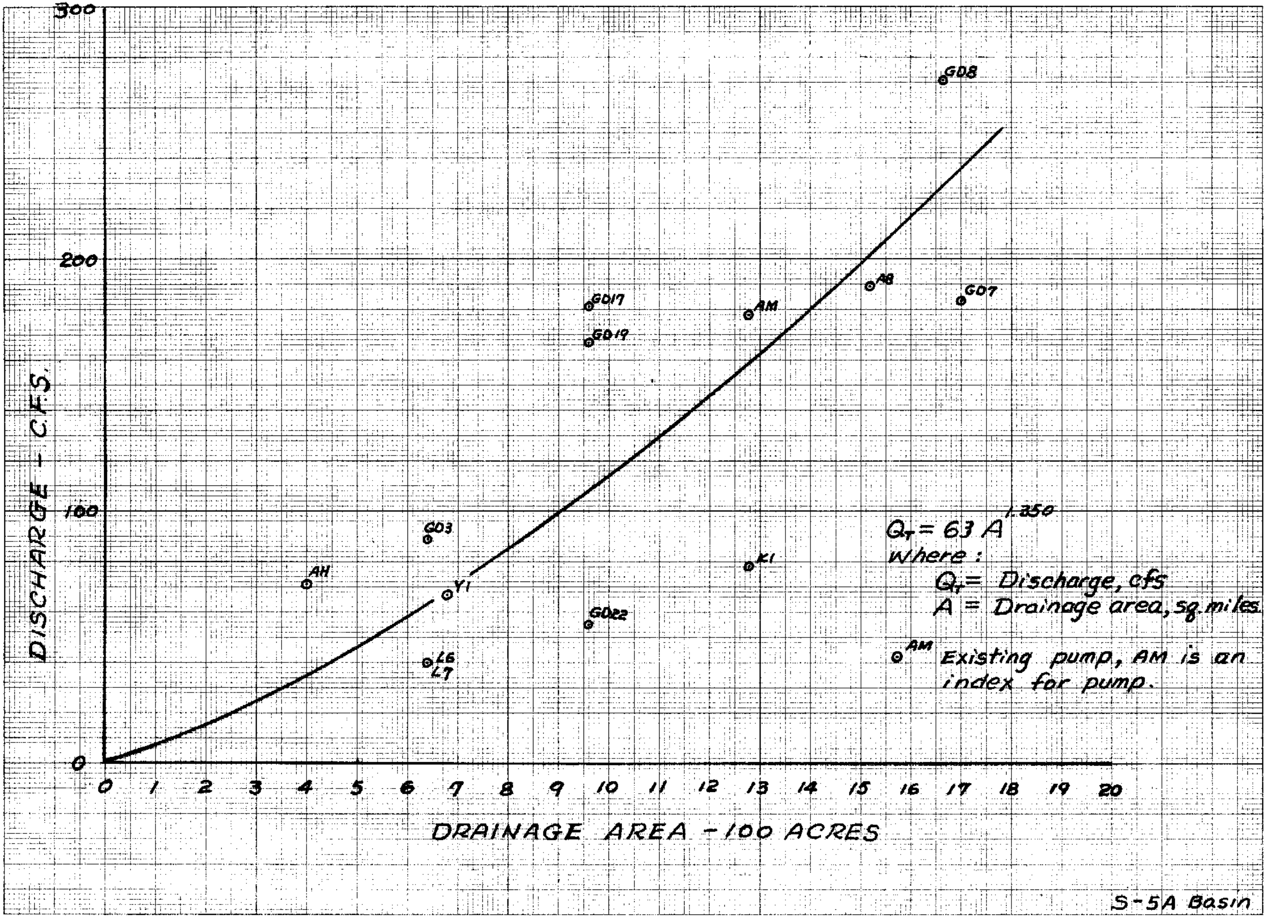


PLATE 30. DISCHARGE-AREA RELATION OF TRUCK FARM AREA PUMPAGE

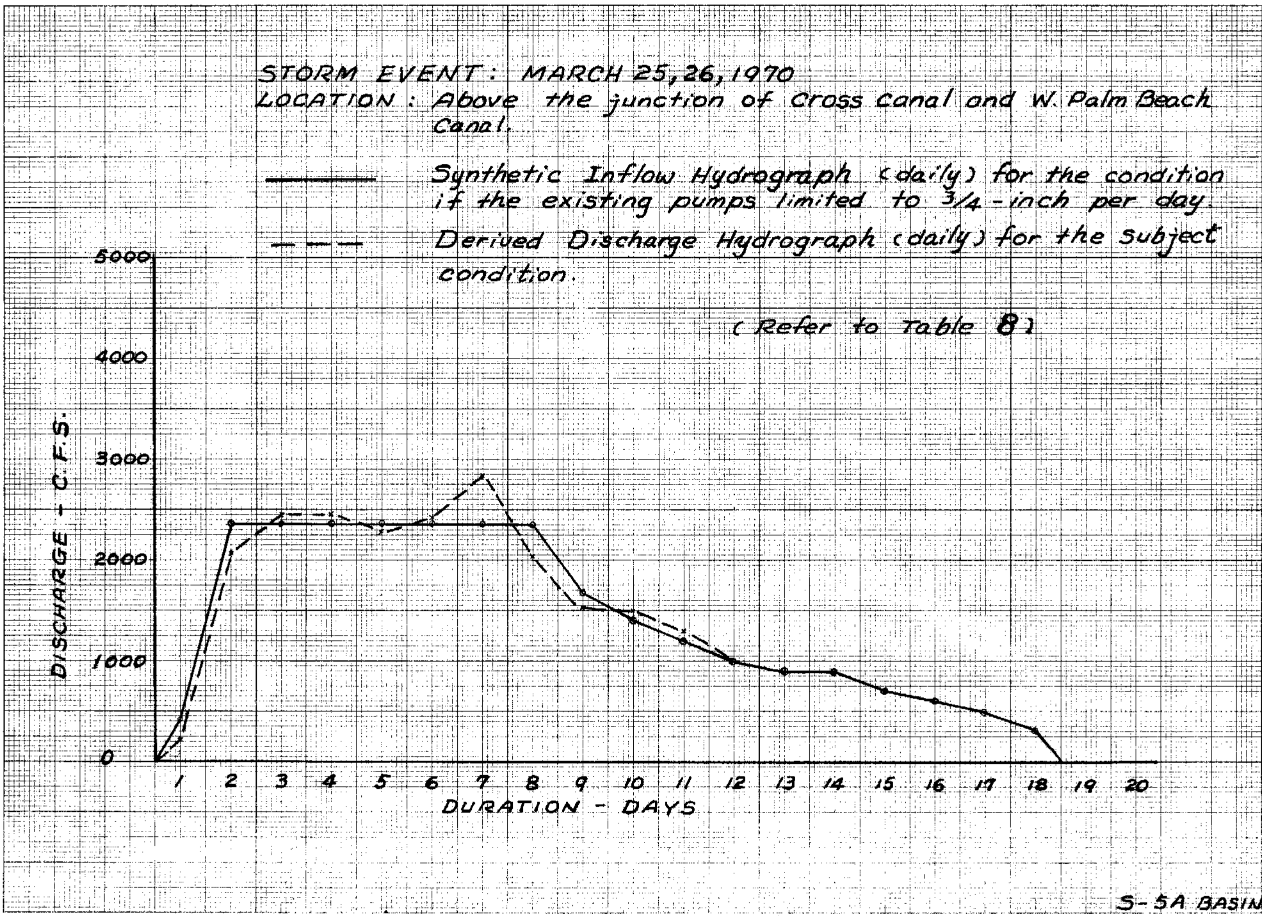


PLATE 31. SYNTHETIC DAILY INFLOW AND OUTFLOW HYDROGRAPHS WITH EXISTING PUMPS LIMITED TO 3/4 INCH PER DAY ABOVE THE

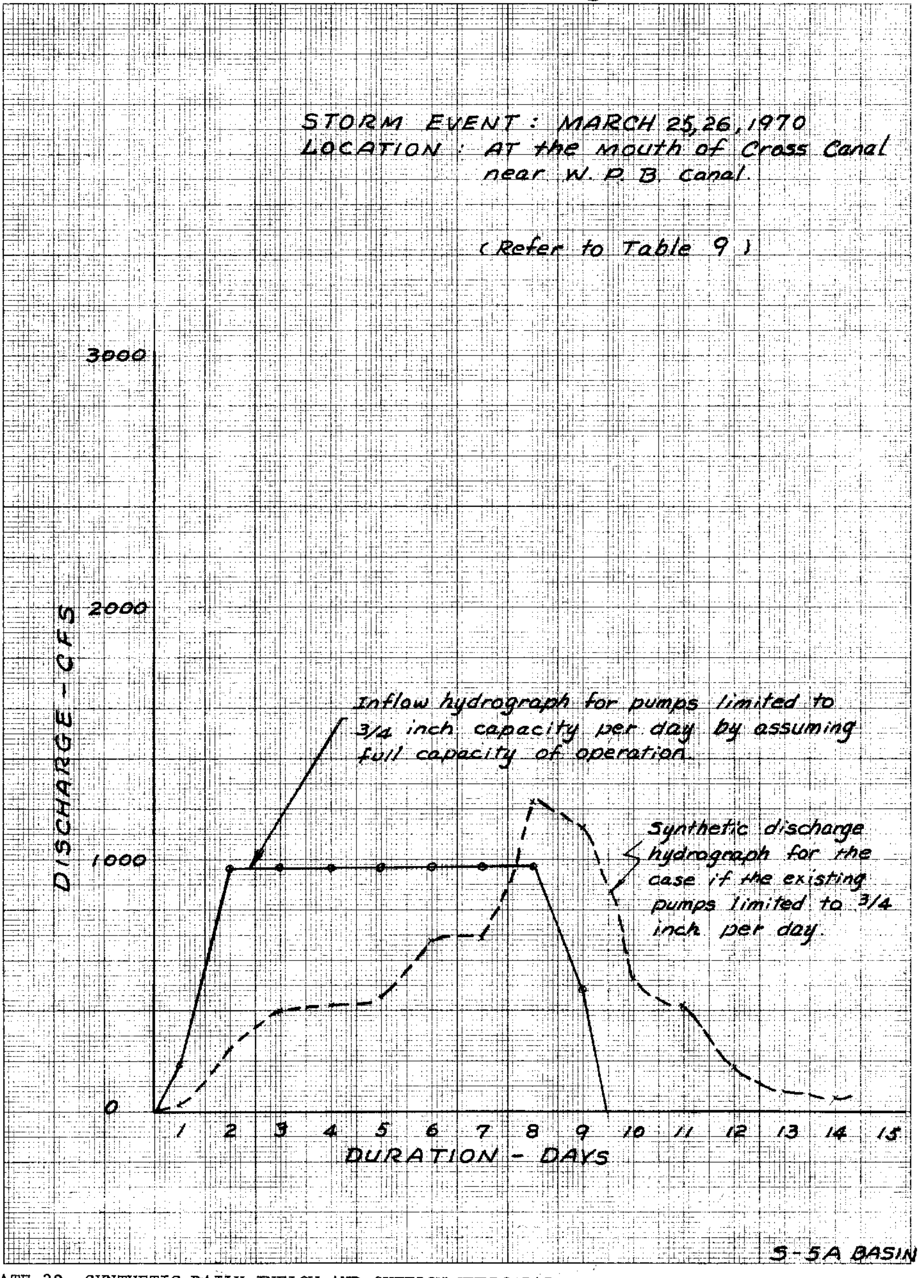


PLATE 32. SYNTHETIC DAILY INFLOW AND OUTFLOW HYDROGRAPHS WITH EXISTING PUMPS LIMITED TO 3/
INCH PER DAY AT THE MOUTH OF CROSS CANAL FOR THE STORM EVENT OF MARCH 25-26, 1970.

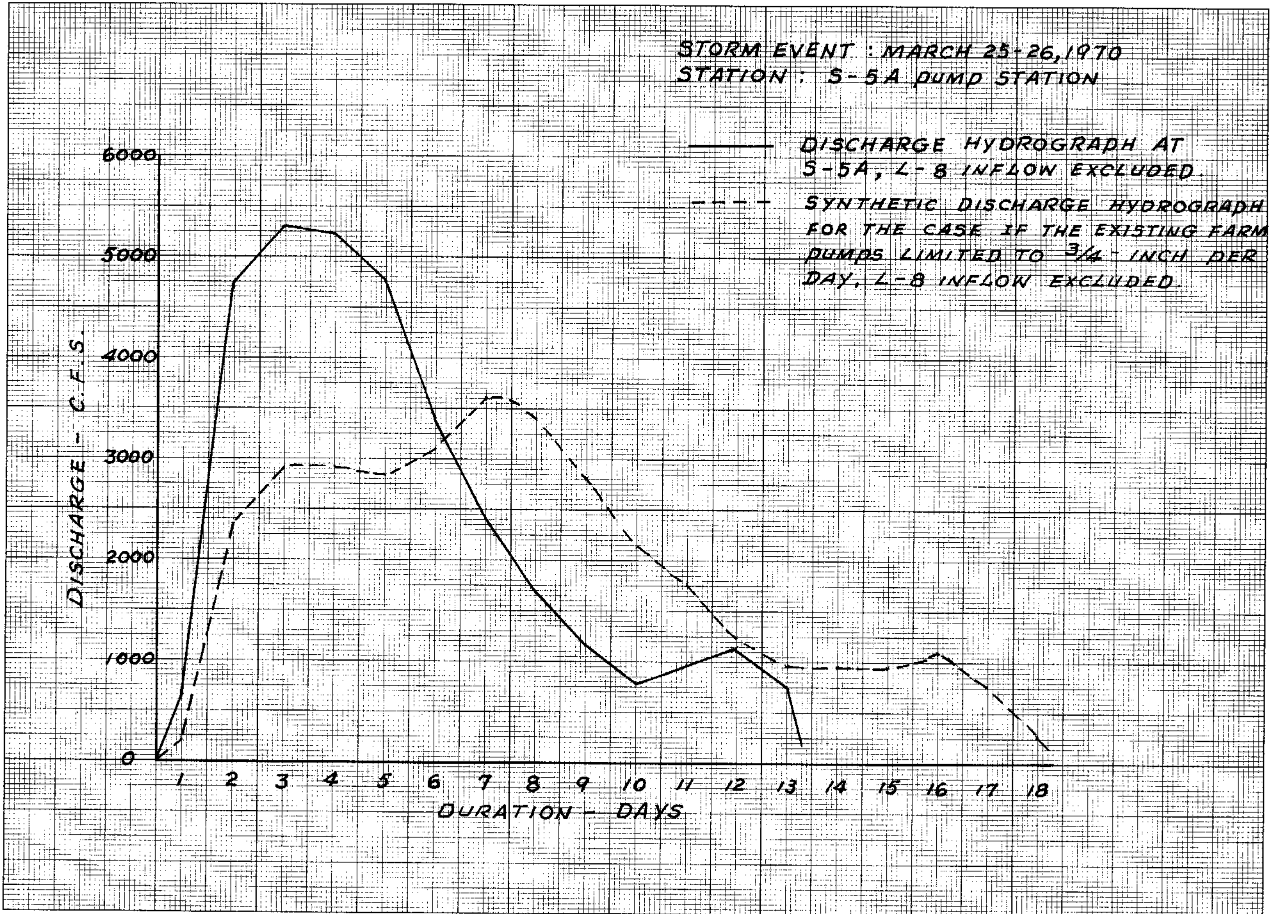


PLATE 33. DISCHARGE HYDROGRAPH FOR S-5A COMPARED WITH A SYNTHETIC HYDROGRAPH WITH EXISTING PUMPS LIMITED TO 3/4 INCH PER DAY

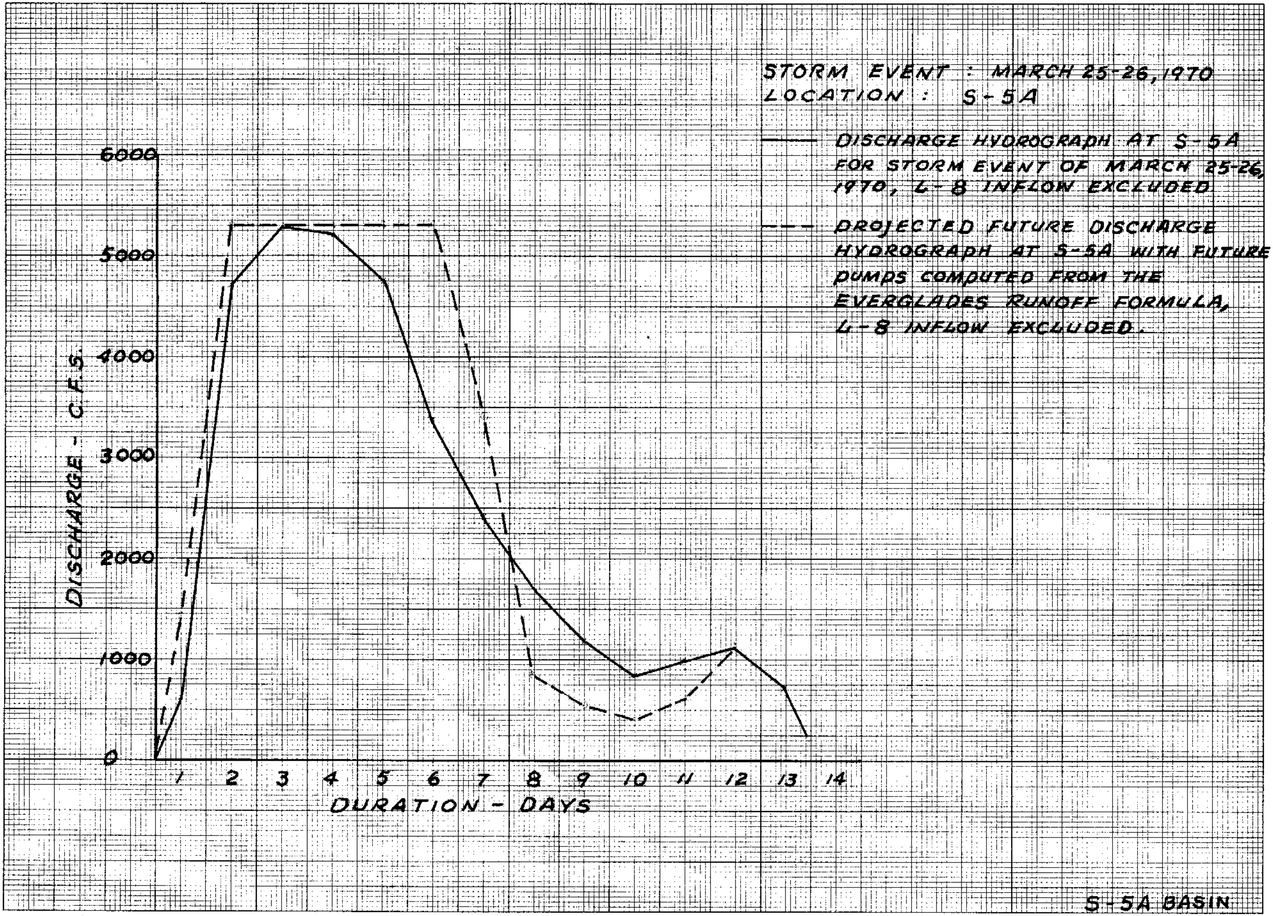


PLATE 34. DISCHARGE HYDROGRAPH FOR S-5A COMPARED WITH A SYNTHETIC HYDROGRAPH WHEN THE ENTIRE BASIN IS DEVELOPED AND

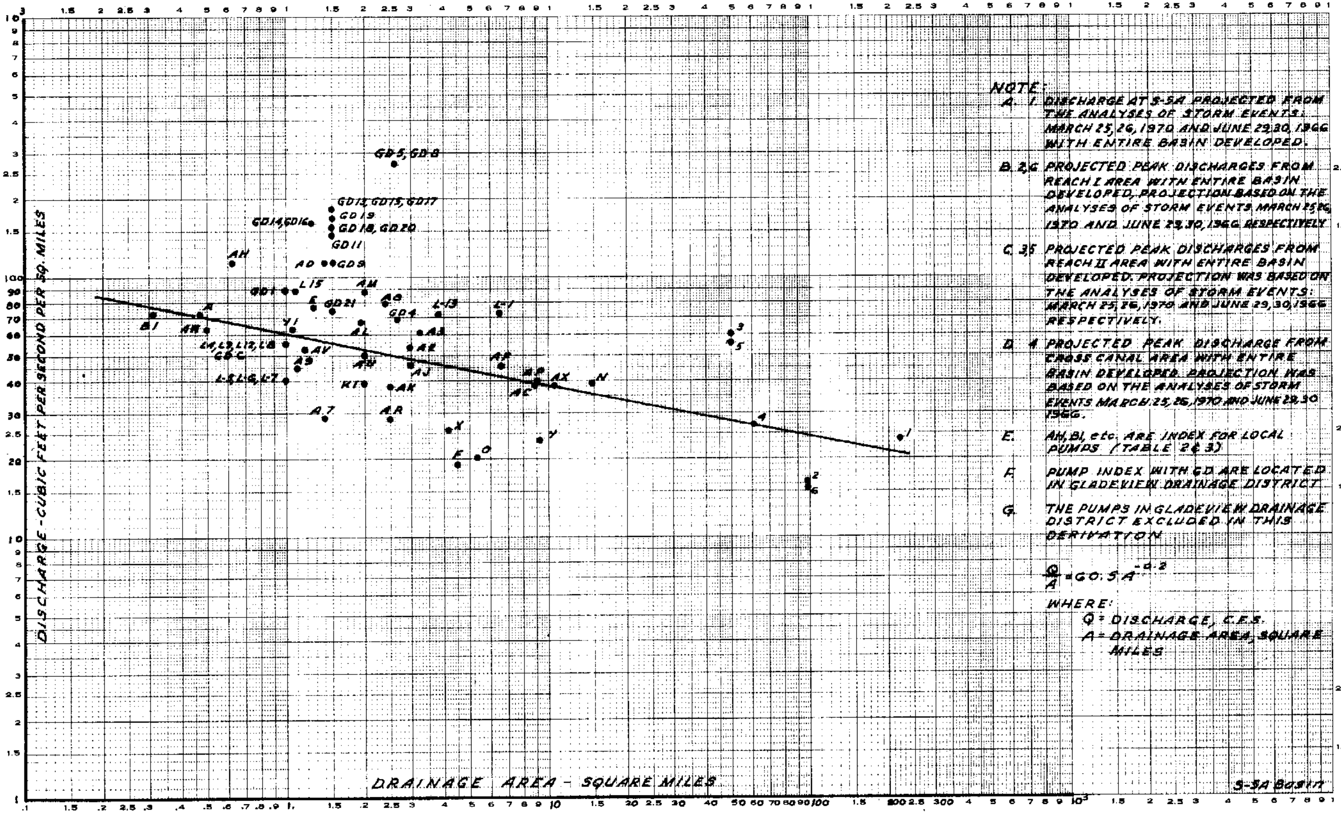
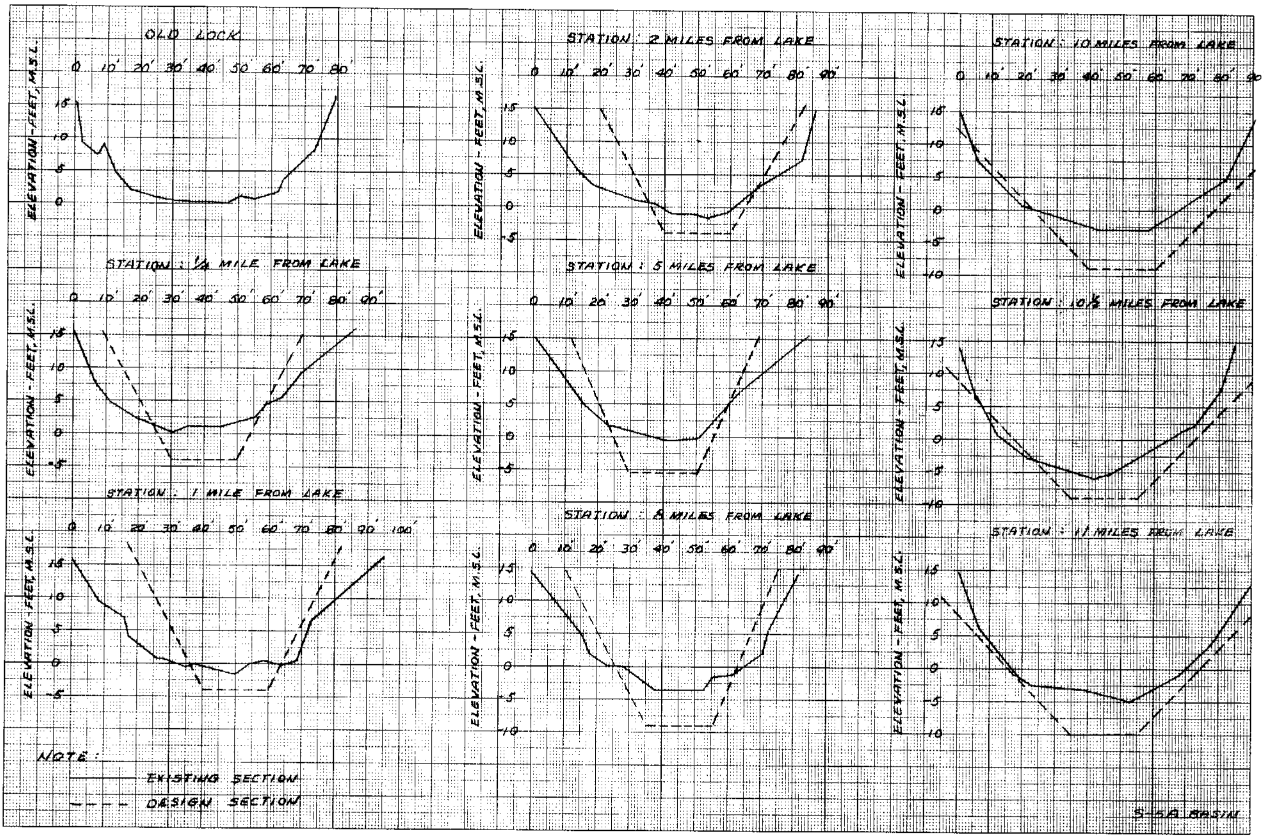


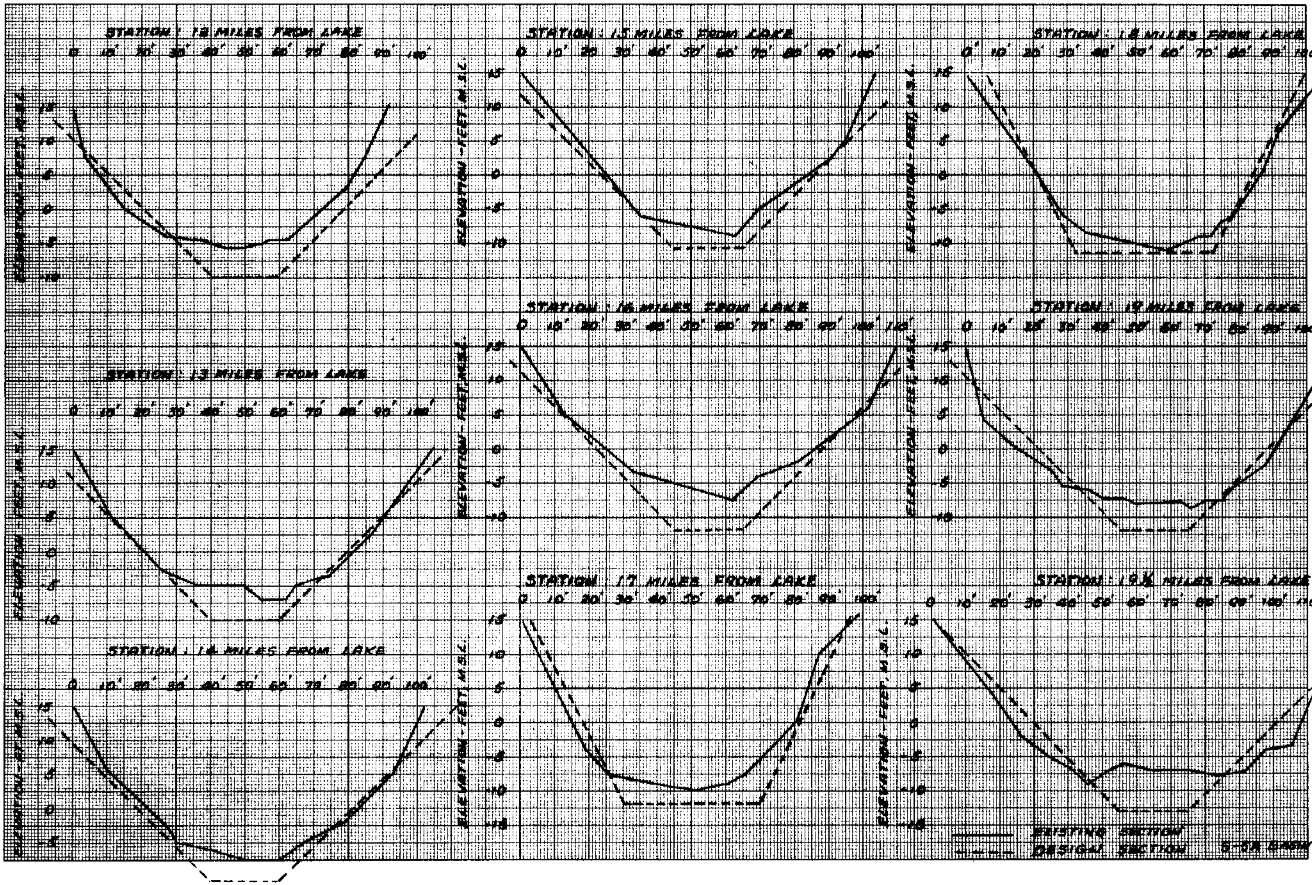
PLATE 35. DISCHARGE — DRAINAGE AREA RELATION WITH ENTIRE BASIN DEVELOPED



APPENDIX A

CROSS-SECTIONS OF WEST PALM BEACH CANAL

SHEET 1 OF 2



APPENDIX A

SHEET 2 OF 2

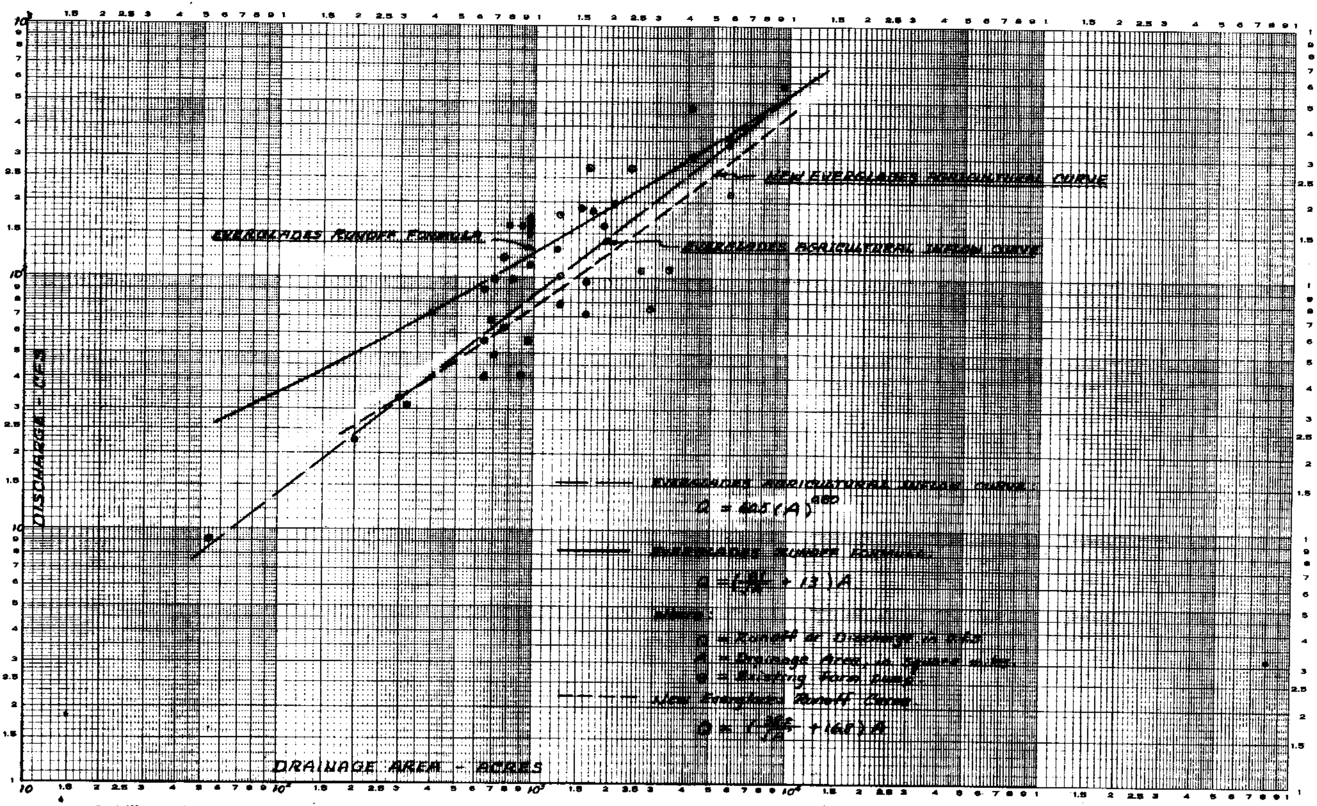


PLATE 27. COMPARISON OF EXISTING FARM PUMPAGE, EVERGLADES RUNOFF FORMULA, AND RUNOFF CURVES.

However, 19.5 square miles of the Cross Canal basin comprises the Gladeview Drainage District which has a present installed pump capacity approximating 1760 cfs. This represents runoff removal capability of 3.34 inches per day. The remaining 41.5 square miles of the Cross Canal basin, with 1580 cfs installed pump capacity, has a runoff removal capability of only 1.41 inches per day.

The Gladeview Drainage District is presently served by thirty-two pumps, the majority of which were installed prior to the March 1970 event. The majority have capacities greater than the presently used Everglades Runoff Formula will allow (Table 3) and for which no permit applications were made.

Installed pump capacity, in inches/day, related to land use in the Gladeview Drainage District is compared with the remainder of the S-5A basin in the following table:

AREA	SUGAR CANE	TRUCK	IMPROVED PASTURE	ENTIRE AREA
Gladeview Drainage District	3.11	3.77	2.07	3.35
Remainder of S-5A Basin	1.57	2.47	1.65	1.94

There is no justification for the pumping capacities which are now in existence in the Gladeview Canal area. They represent a potential which can affect the future performance of the S-5A system. The entire Gladeview area should be treated as a single land unit contributing inflow to the primary system (Cross Canal).

Application of the Everglades Agricultural Inflow Curve to the 19.5 square mile area of the Gladeview Drainage District would produce an allowable inflow of 650 cfs, or an overall runoff removal rate of about 1.25 inches per day. Since the analysis herein indicates that some degree of control is exercised by the dimensions of the Gladeview Canal itself and the highway culvert, a uniformly applied runoff removal rate of 2.0 inches/day would be acceptable.

RECOMMENDATIONS

Based on the findings of this investigation the following recommendations are made for the purpose of maintaining and improving the present satisfactory flood control performance of the S-5A primary system in consideration of the future complete agricultural development of the watershed:

1. Make such detailed surveys of the condition of the L-10 and L-12 channels as are needed to specifically define the nature, volume and location of deposited material.
2. Based on the above surveys, undertake a program of channel cleanout properly phased in terms of need and budgetary requirements over the course of the next two to three years.
3. Immediately adopt and apply the Everglades Agricultural Inflow Curve described herein to future permit applications for allowable discharges.
4. Develop pumping operations procedures for S-5A using the guidelines and criteria outlined herein.
5. Initiate needed action to place pumping installations throughout the watershed, and particularly in the Gladeview area, under permit.
6. Apply the criteria outlined herein for treatment of the Gladeview area secondary pumping installations, taking such action as needed to ensure compliance.

It is further recommended that for the remainder of the Everglades Agricultural Area, the following be done:

1. Perform preliminary primary channel condition surveys.
2. Verify Manning's roughness coefficient values by means of discharge measurements, and revised design water surface profiles if necessary.

3. Develop channel cleanout programs where indicated by the above surveys.

4. Review, and revise as necessary, pumping operations procedures at all locations in the light of the findings of this report.

5. Develop and apply "Everglades Agricultural Inflow" curves to secondary inflow permit applications in all watersheds.

REFERENCES

1. Corps of Engineers, Part I Basic Report, Agricultural and Conservation Areas, dated July 10, 1951.

2. Corps of Engineers, Part I, Supplement 3, Design Memorandum, Pumping Station 5A, dated January 28, 1952.

3. Corps of Engineers, Part I, Supplement 6, Design Memorandum, Gates and Operating Machinery of Control Structures. S-5A-E, S-5A-W, and S-5A-S, dated March 5, 1952.

4. Corps of Engineers, Part I, Supplement 16, Design Memorandum, West Palm Beach Canal (Levess 10 and 12), dated October 15, 1953.

5. Corps of Engineers, Part VI, Section 8, Design Memorandum, Rainfall-excess Evaluation, dated January 5, 1955.

6. Central and Southern Florida Flood Control District, Hydrologic and Operational Report on the December, 1957 Flood in the West Palm Beach Canal Flood Control System Above Pumping Station S-5A, (1958).

7. Galliher, Claiborne F., Water Accounting at Pumping Station S-5A in Florida, Journal of the Irrigation and Drainage Division, ASCE, Vol. 95, No. IR4, Proc. Paper 6970, pp. 517-524.

8. Chow, V. T., Open-Channel Hydraulics, Chapter 8, Section 7.

9. Corps of Engineers, Part VI, Supplement 13, Hydraulic Design of Inlet Structures, Paragraph 7(a), dated June 24, 1960.

10. U. S. Geological Survey, Hydrologic Effects of Water Control and Management of Southeastern Florida. 1972. p. 56.