PHASE I
PROJECT REPORT
TURKEY CREEK WATERSHED

BERKELEY

for

CHARLESTON DISTRICT
OFFICE OF THE CORPS OF ENGINEERS

CONTRACT NO. DACW60-75-C-0011

by

WATER RESOURCES ENGINEERS, INC.

8001 Forbes Place
Springfield, Virginia 22151
703/321-9393

DECEMBER 1975

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WRE

WATER RESOURCES ENGINEERS, INCORPORATED

1 December 1975

Corps of Engineers
Charleston District
Federal Building, Room 429
Charleston, South Carolina 29402

ATTENTION: Ed Meredith

Dear Sirs:

Enclosed are two copies of a final report for Phase I of Contract No. DACW60-75-C-0011, "Preparation of a Detailed Project Report and Environmental Assessment on Turkey Creek, Berkeley County, South Carolina." WRE felt that this report, though not specifically addressed in the contract work scope, was appropriate and we would appreciate your comments on it.

The report represents a detailed and thorough analysis of the hydrologic and hydraulic conditions associated with the Turkey Creek project. The hydrologic information includes all modifications to our original work suggested by your staff. All information concerning the damage survey and the average annual damage levels associated with alternative project designs are also given. This report presents all the work completed in response to paragraphs 1-3 and 5-7 of the contract scope of work and thus corresponds to all items included in Phase I of the contract except the detailed surveying associated with final design alternatives. This item will be completed when the detailed design and costing work associated with paragraph 8 is undertaken.

We believe that the submission of this report represents the fulfillment of all contractual obligations of Phase I with the exception of the detailed surveying mentioned above. In fact, under the specifications of Change Order No. P00004 dated 22 September 1975, which modified the completion date of Phase I to 30 November 1975, WRE is no longer authorized to work on this phase of the contract. We will nevertheless answer any questions pertaining to the attached report and, should it be necessary, hold a meeting with your staff.

8001 FORBES PLACE / SPRINGFIELD, VIRGINIA 22151 / TEL (703) 569 9600

Walnut Creek, California • Springfield, Virginia • Austin, Texas

We would like to point out that we believe it is very important to proceed with Phase II as soon as possible in order to maintain a level of quality consistent with Phase I results. Any large time delays will cause problems with restarting the effort and assigning personnel. For these reasons, we urge you to do everything possible to expedite authorization of Phase II and we would appreciate hearing from you an estimate of when the work might be started.

Sincerely yours,

WATER RESOURCES ENGINEERS, INC.

Robert Taylor

Robert S. Taylor, Jr., P.E.
Senior Engineer

RST/kws
Encl: As described

TABLE OF CONTENTS

1. *PURPOSE.* The purpose of this report is to present the hydrologic and initial hydraulic analysis of Turkey and Goose Creeks, Charleston, South Carolina.

2. *PRIOR REPORTS.* Data from two previous studies are incorporated into this report. These earlier studies are:

a. *Reconnaissance Report.* In March 1972, the Charleston District prepared a reconnaissance study (1)* of flood protection measures along Turkey Creek. This report found a detailed project report was feasible.

b. *Flood Report.* Shortly after June 1973, the Charleston District prepared a report (2) that presented the basic data collected during field investigations of the areas affected by the flood stages that occurred during June 1973.

3. *LOCATION.* Turkey Creek watershed is located in the southern coastal plains region of South Carolina within the Hannahan and North Charleston areas. These areas are contained by parts of Berkeley and Charleston Counties. The Turkey Creek watershed discharges its waters to Goose Creek, which in turn is tributary to the Cooper River. The Cooper River drains directly into the upper limits of the Charleston Harbor navigational channel. A location and prominent features map of the watershed is shown as Figure 1.

4. *WATERSHED DESCRIPTION.* Turkey Creek has a drainage area of about 4.06 square miles and an unimproved length of about 3.9 miles. The watershed is located in a moderately urbanized area consisting primarily of single family residential dwellings in the lower reaches and mobile homes and commercial establishments in the upper portion of the watershed. For hydrologic considerations, the area of study coincides with the drainage boundaries of Turkey Creek and for backwater computations the detailed channel investigation begins 740 feet upstream of Interstate 26 (▲ on Figure 1) and terminates at the confluence of Turkey Creek with Goose Creek. For study purposes, Turkey Creek is subdivided into three major reaches. Reach 1 begins at the mouth and extends upstream to the Seaboard Coast Line Railroad (SCLRR) embankment. Reach 2 begins at the SCLRR embankment and extends upstream to the U.S. Highway 52 crossing. Reach 3 begins at the U.S. Highway 52 crossing and terminates at the upstream study boundary, a point approximately 740 feet upstream of U.S. Interstate Route 26.

*Numbers in parentheses refer to references listed at the end of this report.

Average stream slope is 6.6 feet per mile. Ground elevations in the watershed change gradually from 2 feet mean sea level (msl) at the mouth to 49 feet msl at the upstream study boundary. The elevation of the existing channel bottom varies from -5.8 feet msl at the mouth to 11.2 feet msl at the upstream model limits. The bottom of the existing channel is below mean high tide (elevation 2.4 feet msl) from the mouth to a point just downstream of the Seaboard Coast Line Railroad crossing. Channel top widths vary from 50 feet at the mouth to 5 feet in the headwaters.

5. *TIDAL EFFECTS*. Tidal effects on Turkey Creek are pronounced and vary with the force, direction, and duration of winds and other meteorological events occurring seaward. Generally, tidal stages in Turkey Creek have a departure of 1.5 to 2.0 feet above normal tide experienced in Charleston Harbor and a lag time of about 3 to 4 hours. Normal tides in Charleston Harbor range from -2.6 feet msl to 2.6 msl with a mean spring high tide of about 4.2 msl.

6. *STREAM CROSSINGS*. In the area of study for channel improvement Turkey Creek is bridged by five roadways and is spanned by one double track railroad embankment. Beginning with the upstream crossing, and proceeding downstream, these crossings are discussed below:

a. *U.S. Interstate Route 26*. This dual highway links with several other interstate routes in North Carlolina to provide a corridor for shipment of goods and commerce from northern and mid-western states to the seaport of Charleston. It also serves as a local commuter route between Charleston proper and its outlying communities,

b. *J.M. Fields Access Road*. This bridge provides local traffic with a secondary access route to the J.M. Fields department store. Traffic volume is modest and is limited to that generated by local area residents.

c. *U.S. Highway 52 Crossing*. Route 52 is known by local inhabitants as Rivers Road. It is conveyed across Turkey Creek by embankment fills. At the crossing site it is dual-laned and divided. The highway itself supports many functions. It is a heavily traveled route that interconnects the outlying areas with downtown Charleston. In addition, it supports a variety of commercial activities which serve local residents.

d. *Hawthorne City Bridge*. Turkey Creek flows through the center of Hawthorne City. The primary function of this crossing is to connect the north and south extremities of the trailer park and to provide local residents with access to neighboring through routes.

e. *Seaboard Coast Line Railroad (SCLRR) Embankment.* The SCLRR embankment fill carries a double track railroad across Turkey Creek and provides for the movement of bulk materials and goods in and out of Charleston Harbor.

f. *Murray Avenue Bridge.* Murray Avenue crosses Turkey Creek near its mouth and is primarily a commuter route providing the residents of Hanahan City and surrounding locale with access to the work centers in and around downtown Charleston.

7. *STREAM FLOW DATA.* There are no streamflow gaging records or stations for Turkey Creek nor are there any within the immediate vicinity. Streamflow measurements are limited to a single measurement of 866 cfs recorded near the peak of the flood of June 11, 1973, at the Murray Avenue crossing. Streamflow measurements are also nonexistent for Goose Creek. Discharge data for these watersheds have been synthesized and are presented in subsequent sections of this report.

8. *FLOODING.* Flooding problems along Turkey Creek have increased in recent years because of continuing urbanization of the watershed. Commercial establishments have recently located in the area and have paved several acres as parking areas. As a result, the time of concentration of runoff has been decreased and the infiltration capacity of the watershed has been diminished, thus producing higher peak flows from intense rainfall associated with storms of short duration. Tidal fluctuations in the lower reaches further complicate the natural drainage of the basin by producing a backwater effect which causes flood waters to achieve a higher stage.

9. *PRECIPITATION.* A National Oceanic and Atmospheric Administration (NOAA) precipitation station is located within the Turkey Creek watershed at Charleston Municipal Airport, elevation 40 feet msl, latitude 32°54' North, longitude 80°02' West. Precipitation data have been measured for the Charleston area since 1871. Data recorded by this station during the calibration period, June 1973, are presented in Table 1. Rainfall recorded over this watershed during June 1973 established a new one-day record. The previous one-day record was 8.84 inches recorded in September 1945. Rainfall recorded on the evening of 19 June 1973 also created flood problems.

A three-hour storm dumped 4.58 inches of rain in the watershed with approximately 4.5 inches falling within a 90-minute period. The record rainfall recorded during the period of 8-12 June 1973 resulted from a weak low pressure trough with insufficient winds to carry the high moisture clouds away from the area. Rains began falling around noon (EST) on the 8th of June and dumped a total of 4.61 inches on that day. Rains continued during the 9th and 10th but not as intense. Heaviest rainfall was experienced on 11 June when a total of 9.40 inches was recorded. During the 90-hour period from 11:00 am on the 8th to 9 pm on the 12th, approximately 17.0 inches of rain fell. The unadjusted standard project storm rainfall index for this region is about 14.7 inches; adjusted for areal distribution it becomes about 20.6 inches. Thus, the rains of the 8-12th of June approximate the standard project storm in both magnitude and duration. Rains recorded on the 11th were approximately equivalent to the 50-year 24-hour rainfall as predicted in the Weather Bureau Technical Paper No. 40, *Rainfall Frequency Atlas of the United States* (3).

10. FLOOD DAMAGES. During the calibration period, June 1973, Turkey Creek overflowed its banks on three separate dates. The area was inundated on 8, 11, and 19 June 1973. The flood of the eleventh achieved the highest stage and produced the greatest damage. On the morning of 9 June 1973, the stage from the previous day's flood had subsided but the rain continued throughout this day and the next. On the following day, June 11th, torrential rainfall established a new daily rainfall record of 9.40 inches. Flood waters began to rise and surpassed those recorded on the 8th and 9th. Residences in the lower reaches and mobile homes in the upper reach were inundated by these waters.

Near peak flood stage the stream discharge was measured to be 866 cfs with a mean velocity of 1.77 feet per second. Turkey Creek was experiencing a mean tide at the time the flood of the 11th peaked. Had the tide been at a higher elevation flood damage would have been more extensive. High water elevation from this flood were recorded and a plan view of the flooded area is presented in Figure 2.

During recent field investigations it was noted that one of the culverts under the SCLR embankment was heavily silted. Persons familiar with the study area confirmed the suspicion that this culvert was in the same condition during the June 1973 flood. Flood stages in the upper reaches would not have been as severe had this culvert functioned more effectively. Also, the highwaters in the trailer park deposited large amounts of silt. This silt was reported to come from land laid bare when new commercial facilities were constructed in Grant City North and from erosion of the Seaboard Coast Line Railroad embankment. The erosion problem is under investigation by the local office of the U.S. Soil Conservation Service (SCS). It is envisioned the SCS will cause landowners in this vicinity to effect remedial conservation practices to abate land erosion. These practices will have a significant impact on maintenance requirements for Turkey Creek under future conditions. For study purposes, it has been assumed that the deposition of excessive silt will be eliminated.

11. *SYNTHETIC FLOWS.* Since there is only a single discharge measurement for Turkey Creek, and none for Goose Creek, it was necessary to use synthetic means to determine rates of discharge for these streams. Two independent methods of forecasting were selected. Discharge frequency curves were developed utilizing both rainfall-unit hydrograph techniques and a regionalized frequency analysis based on statistical analysis. Following these independent determinations of discharge frequencies, the results were adjusted to common values. Discussions of the rainfall-unit hydrograph technique, regional frequency analysis, and reconciliation of results are presented in the sections that follow.

12. *UNIT HYDROGRAPH ANALYSIS OF TURKEY CREEK.*

a. General. The development of a rainfall-runoff relationship for this basin evolved through a series of sequential steps. First, synthetic unit hydrographs were developed for three locations within the Turkey Creek basin. These three locations were: at the mouth of Turkey Creek; at the mouth of the unnamed tributary to Turkey Creek; and at the site of the SCLRR crossing. Flood flow data from the June 1973 storms (see Paragraph 7) was then used to calibrate the unit hydrographs. After the unit hydrographs were calibrated, they were utilized to compute runoff from storms having 2-, 10-, 25-, 50-, and 100-year return frequencies. Each of the major steps undertaken during these calculations is described in the paragraphs that follow.

b. Unit Hydrograph Parameter Section. Prior to the computation of synthetic unit hydrographs, the following physical characteristics were determined for each subbasin of interest:

- L river mileage from a given station to the upstream limits of a subbasin drainage area
- L_{ca} river mileage from the station to the center of gravity of the subbasin drainage area
- DA drainage area of the subbasin in square miles
- S_{st} mainstream slope of the longest water course in the basin

Values for these parameters were obtained in accordance with the guidance given in EM 1110-2-1405 (4). A relationship between the physical characteristics of a subbasin and the two unit hydrograph parameters listed below was sought:

- t_r lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours
- q_p peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cubic feet per second (cfs) per square mile

Corps District offices in the Baltimore, Norfolk, Wilmington, Charleston, Savannah, Jacksonville and Mobile Districts were asked to provide any available unit hydrograph analysis of studies that had been completed on basins similar to Turkey Creek. The information received, along with data recorded in two volumes of *Civil Works Investigations* (5, 6) for 6-hour unit hydrographs was plotted on log-log paper. Plots of t_p versus LL_{ca}, $(LL_{ca})^{0.3}$, (LL_{ca}), $/\sqrt{S_{st}}$, $L\sqrt{S_{st}}$, and DA were generated as well as plots of q_p versus t_p and DA. These plots were then fitted with a straight line and the equation of the line was calculated. Thus a regionalized functional relationship between unit hydrograph parameters and subbasin characteristics was determined. Only five of these plots showed a high degree of correlation, three of which are shown on Figures 3, 4 and 5. By judgmental weighting of the plots, initial estimates of t_p and q_p were made. These values were then adjusted to 1-hour unit hydrograph values and utilized as a preliminary value for computing the Snyder coefficients of C_t and 640 C_p.

c. Rainfall Losses. Surface soils in Turkey Creek watershed vary from fine sandy loam to sandy clay loam in the upper reaches to clayey sandy loam in the downstream reaches and the flood plain. The subsoils in the watershed range from plastic fine sandy loam to fine sandy clay. These soil types normally have moderate infiltration rates and are moderately well-drained; however, the presence of a permanent high water table and the high swelling potential of the clays serve to give these soils a high runoff potential.

Standard infiltration curves for each of the four hydrologic soil groups classified by the U.S. Soil Conservation Service have been developed by the State of Illinois (7). For the soil types described above, initial and final constant infiltration rates of 5.0 and 0.20 inches per hour, respectively, are reported. These rates are given as maximum values which require adjustments for seasonal variability, as well as antecedent conditions.

Values of 1.2 and 0.20 inches per hour for initial and final constant rainfall losses, respectively, were selected for use in calibrating the HEC-1 model with the June 1973 rainfall events. More critical values of 1.0 and 0.05 inches per hour were used in conjunction with the 2-, 5-, 10-, 25-, 50-, and 100-year return rainfall events and the standard project storm rainfall to maximize storm runoff.

d. Model Calibration. The computed Snyder coefficients were used with the HEC-1 computer program, Flood Hydrograph Package, in an attempt to reproduce the June 11, 1973 flood flows. The effort was unsuccessful.

From a consideration of the June flood flow with that predicted by the initial HEC-1 model, it was evident that the storage utilized during this flood flow had an influence on model results. Accordingly, the Turkey Creek watershed was divided into three subbasins and a storage-outflow relationship was developed so that flood flows could be routed from the SCLRR to Murray Avenue using a modified-Puls method. Much better model results were achieved with the model capturing both the peak flows and the time to peak as measured during the June 1973 storm. However, storage utilized by the second model was much in excess of that of the June 1973 flood. This necessitated further model refinement.

The effect of the culverts at the SCLRR crossing on flood flow regime led to the adoption of a third model shown on Figure 6. The SCLRR route model is a fairly sophisticated representation of the physical characteristics of the watershed. Storage-outflow relationships were developed for the reach between the unnamed tributary and Murray Avenue, the reach between Murray Avenue and the mouth of Turkey Creek, and at the site of the SCLRR culverts. Special emphasis was placed on determining the storage-outflow relationship at the SCLRR culverts. The hydraulic performance of these culverts under flood conditions was evaluated using known culvert geometrics and slope, known high water marks during flood stage, and the criteria established for culvert hydraulics as contained in the *Virginia Department of Highways Drainage Manual* (8). The computed discharge rating curve for the nonsilted culvert is shown as Figure 7. The twin culvert is heavily silted and was assumed to be only one-third as hydraulically effective as the nonsilted culvert. This assumption prevailed during model calibration and during the determination of discharges for the preselected 2-, 5-, 10-, 25-, 50-, and 100-year return storm events. The discharges through these culverts for the standard project storm were determined by assuming that both culverts were open. Justification for this latter assumption is based on the scouring action produced by the higher velocities of the standard project storm waters.

This third system model yielded good agreement between observed and predicted flood values. The model predicted a peak flow of 876 cfs would occur at the Murray Avenue bridge between 11:00 and 12:00 am on June 11, 1973. A flood discharge of 866 cfs was measured at 11:45 on the same date at this location. Further agreement was obtained with respect to flood storage volumes. The model predicted flood waters would require 152 and 111 acre-feet of storage between the SCLRR crossing and Murray Avenue, and between Murray Avenue and the mouth of Turkey Creek, respectively. An approximation of the storage actually utilized was determined by planimetering the flood profile map, Figure 2, between these points and multiplying the calculated surface area by the average end area of surveyed cross-sections taken between the points. These calculations yielded values of 190 and 120 acre-feet, respectively.

Table 2 lists subbasin and unit hydrograph data employed in the adapted model. The end of period unit hydrograph ordinates for each subbasin of the model are shown in Table 3. Data for a Goose Creek subbasin also appears in these tables and will be discussed in a subsequent section.

e. *Selection of Rainfall Duration.* Rainfall values for selected return periods and durations were determined by utilizing the data presented in *Weather Bureau Technical Paper No. 40*. These values are shown in Table 4. Before these values were used to compute the discharge frequency curve for Turkey Creek, it was necessary to choose a specific duration. Initially, it was thought that adequate protection could be gained by utilizing a duration that approximated the time of concentration at the mouth of Turkey Creek. Since the time of concentration is about 5 hours, this would have led to the selection of a 6-hour rainfall duration. A sensitivity analysis utilizing the adapted model and the 6- and 24-hour duration rainfall events showed that there was little difference in the peak outflow between the 6- and 24-hour duration events. However, the 24-hour duration events required much more natural storage than did the 6-hour. This suggests that damages from the 24 hour duration events would be much greater since flood stages would be more pronounced. Accordingly, the 24-hour duration events were selected as a basis for design.

f. *Time Distribution of Rainfall.* To accomodate a 1-hour unit hydrograph analysis the 24-hour duration rainfall was first subdivided into four 6-hour time periods and then each 6-hour period was further subdivided into hourly increments. The time distribution of rainfall was made in a manner similar to that described in *Civil Engineer Bulletin No. 52-8* (9). Specifically, subdivision into 6-hour time periods was made by applying the estimates shown at Plate 10 of reference 9. Further subdivision of each 6-hour time period into hourly values was made by selecting the hourly distributions shown at Plate 11, reference 9. Table 5 lists the data selected from Plates 10 and 11 and the hourly percentages of total 24-hour rainfall. These latter values were determined by multiplying the percentage of 24-hour rainfall in each 6-hour period by the 1-hour rainfall expressed as a percentage of total 6-hour rainfall.

The hourly percentages of total 24-hour rainfall thus determined were then used to distribute the total 24-hour rainfall for selected return frequency events. Hourly rainfall values for these selected events are shown at Table 6.

g. *Calculation of Runoff*. Peak runoff from each of the preselected return frequency storms was determined by applying the distributed storm rainfall to the calibrated unit hydrographs and calculating rainfall excess with the HEC-1 computer program. The peak discharges so determined have been plotted as point values on Figures 8, 9, and 10 for the locations of Turkey Creek at its mouth, Turkey Creek at the SCLRR crossing, and the unnamed tributary at its mouth, respectively.

h. *Standard Project Storm*. A standard project storm was computed for Turkey Creek utilizing the previously referenced Engineer Bulletin 52-8 and HEC-1 computer program (Method 2) (10). A total rainfall of 21.1 inches fell over the watershed for this 96-hour duration storm which produced peak outflows of 1354, 487, 1492 and 1536 cfs at the SCLRR crossing, mouth of the unnamed tributary, Murray Avenue, and mouth of Turkey Creek, respectively.

The standard project storm isohyetal pattern was aligned to maximize the rainfall intensity over both the Turkey Creek basin and that part of the Goose Creek watershed above the reservoir. The areas under the concentric isohyetal ellipses were integrated to determine the total percent of standard project storm index rainfall falling over the subbasins in Turkey and Goose Creeks. It was calculated that each subbasin in Turkey Creek would receive 140 percent of the standard project storm index rainfall, while the one Goose Creek subbasin studied would receive 113 percent.

i. Areal Distribution of Rainfall. In most hydrologic studies the analyst determines the areal distribution of rainfall by examining rainfall data from weather stations that bracket his study area. There is only one recording station in the vicinity of the Turkey Creek study area, hence this method could not be used. An alternative method of relating total percents of standard project storm index rainfall was used. From the preceding paragraph it can be seen that the maximum percentage of SPS index rainfall occurring over Turkey Creek is 140 percent and that over Goose Creek above the reservoir is 113 percent. This relationship is utilized to determine the areal distribution of rainfall from all lesser storm events by the following expression:

$$R_{GC} = \frac{113\%}{140\%} R_{TC}$$

that is

$$R_{GC} = 80\% \, R_{TC}$$

where:

R_{GC} = total rainfall over the Goose Creek subbasin for a given rainfall event

and,

R_{TC} = total rainfall over Turkey Creek for a given rainfall event

Thus, it is assumed that for any rainfall selected from Table 4, Rainfall-Frequency-Duration, for Turkey Creek that 80 percent of the same value will fall simultaneously over the Goose Creek subbasin.

13. *UNIT HYDROGRAPH ANALYSIS OF GOOSE CREEK*

a. General. An analysis of the hydrologic behavior of Goose Creek is necessary so that the impact of its flow regime under flood conditions can be assessed with respect to flood stages on Turkey Creek; conversely, the effect of the flow regime of Turkey Creek can be analyzed with respect to flood stages on Goose Creek. Secondarily, the effect of tidal stages on Turkey Creek cannot be considered unless a concurrent evaluation is made of tidal stages on Goose Creek.

This section addresses the manner in which inflow and outflow unit hydrographs were determined for the Goose Creek Reservoir. An inflow unit hydrograph is determined by relating subbasin physical characteristics to unit hydrograph parameters and then relating the unit hydrograph parameters to Snyder coefficients, which in turn are utilized to produce a synthetic hydrograph. This method is discussed in detail in the preceding section.

Before the outflow hydrograph can be synthesized, a storage-outflow relationship for the reservoir must be determined. No discharge measurements or discharge-rating curves exist for the reservoir; therefore, stage-storage and stage-discharge relationships must be developed and related to one another to define the stage-outflow curve. Once this relationship is determined it is used to produce an outflow hydrograph. The inflow and outflow hydrographs are then calibrated with selected storm events. The calibrated model is then used to produce a peak discharge frequency curve. Subsequent paragraphs develop the analysis of Goose Creek reservoir. This section concludes with the calculation of inflow and outflow hydrographs for a standard project storm centered over the Turkey Creek watershed.

b. Inflow Hydrograph. An inflow hydrograph is produced by relating subbasin characteristics to unit hydrograph parameters by the method discussed in paragraph 13b. These unit-hydrograph parameters are employed to calculate Snyder coefficients. The Snyder coefficients are then used in conjunction with computer program HEC-1 to produce a synthetic unit hydrograph. The unit hydrograph parameters and subbasin characteristics are listed in Table 2 and the adopted unit hydrograph end of period ordinates are cited in Table 3.

c. *Stage-discharge determination.* The determination of a stage-discharge relationship begins by considering the geometry of the dam (see Figure 11).Surplus water from the reservoir discharges over two spillways of fixed elevation. The short spillway is at elevation 10.5 feet Charleston Public Waterworks datum (CPW) (Zero of the CPW datum = -3.0 feet msl). The long spillway is at elevation 11.0 feet CPW datum.

The dam embankment cross-section was modified to simplify the calculation of discharges from the weir equation. A triangular cross-section was assumed by determining an equivalent slope for the two overflow face slopes. An equivalent slope is calculated for both the long and short spillways.

In the *Handbook of Hydraulics*(11)the author discusses the calculation of discharges from weirs that have triangular cross-section.

Discharge is computed from the weir equation:

$$Q = CLH^{3/2}$$

where:

L = the effective length of the weir in feet

H = the head above the weir in feet

and,

C = the weir coefficient which varies both with the slope of the downstream face and the head.

In this reference, the author gives tables for values of the weir coefficient for different downstream face slopes and for varying heads. The weir equation is solved by use of these tables for discharges over each spillway. By entering the table with the equivalent slope one can determine the weir coefficient by assuming a head. This value is used to solve for the discharge. The process is repeated by assuming a new head and is continued until discharges have been computed over the range of interest. The water surface elevation that must exist to produce these heads on the spillways is determined by adding the head to the elevation of the flow line for each spillway. Flow line elevation of the short spillway is 10.5 feet CPW datum and of the long spillway 11.0 feet CPW datum. The total discharge over the reservoir for a given elevation is obtained by adding the discharge of the long spillway at that elevation (head plus flow line elevation) to the discharge of the short spillway for the same elevation. The stage-discharge curve thus determined is shown on Figure 12 as "outflow uncorrected for losses." During calibration it was necessary to modify this curve; the final adopted stage-discharge curve is shown as "outflow corrected for losses."

d. Stage-Storage Determination. A stage-storage curve was constructed to determine an outflow-storage relationship for use with computer model HEC-1. Such a curve was prepared with the aid of Table 7 which was extracted from the *Thirtieth Annual Report, Commissioners of Public Works* (12). Storage in millions of gallons was converted to storage in acre-feet. It can be seen from viewing Figure 13 that the curve was extrapolated beyond elevation 11.4 feet, CPW datum (8.4 feet, msl). This extrapolation was based on a consideration of the topographical relief of the subbasin above the spillway.

e. Calibration of Outflow Hydrograph. The Goose Creek reservoir discharge was calibrated as described below. Table 8 is representative of the data that existed for five selected storm events. The August 1940 storm had the greatest magnitude and was selected as the calibration storm.

From the data listed in Table 2, the daily rainfall from Table 8, and a provisional storage-outflow relationship developed from Figures 12 and 13, inflow and outflow hydrographs were computed by applying the HEC-1 computer model. The variation in storage was computed from the difference of these hydrographs, and compared to the variation reported for the selected storm events. This procedure was reiterated, slowly changing the provisional storage-outflow curve, until reasonable agreement between the predicted daily variation in pool stage and actual reported variation was obtained. The results of calibrating the HEC-1 model with the August 1940 storm are shown in Table 9. Two other storm events were modeled with similar results (absolute value of the average percent difference less than 10%).

f. Storage-Outflow Determination. The final adopted storage-outflow curve used in the model is shown as Figure 14.

The difference in outflow predicted by the theoretical weir equation and that predicted by the calibrated model is probably attributable to the loss in weir effectiveness caused by the floating islands of vegetation that block the spillways and the deteriorated condition of the spillways themselves.

g. Peak Discharge Frequency Curve. After the model was calibrated, it was used to forecast both peak inflow and outflow discharges for select return frequency rainfall events. Outflows of 1214, 2653, 3151, 3802 and 4431 cfs were predicted by the model for the 2-, 5-, 10-, 25-, 50-, and 100-year return rainfall events, respectively. These data were not incorporated into a peak outflow frequency curve for several reasons. Most importantly, such a curve could be subject to misinterpretation and subsequent misuse. Secondly, the outflows are representative of the discharges expected to occur when a storm simultaneously occurs over both Goose and Turkey Creek subbasins. Considerably higher outflows from the reservoir would be experienced if consideration were not given to maximizing storm runoff over the latter basin as was done during this study. Lastly, a meaningful regional frequency analysis of the Goose Creek subbasin cannot be undertaken to give validity to the rainfall-runoff discharges because of the detention effect of the reservoir.

h. Standard Project Storm. A standard project storm with the storm center over the Turkey Creek Watershed was computed for this subbasin using procedures previously described. A total rainfall of 16.8 inches was computed for this 96-hour duration event. The standard project storm as centered over Turkey Creek produced a peak inflow of 12,800 cfs and a peak outflow of 8730 cfs for Goose Creek reservoir.

14. REGIONAL FLOOD FREQUENCY ANALYSIS

a. Statistical Frequency Analyses. Statistical frequency analyses of the annual flood peaks from gaging stations in the South Carolina coastal region near Charleston were performed using the *Pearson Type III* distribution (13) to derive regional relationships for the mean, standard deviation, and skew of the flood frequency distribution. The available data consisted of the annual maximum peak flows and watershed characteristics of 21 stations in the region. The means, standard deviations and skews of the logarithms of annual peaks were compiled and computed by the Charleston District office, Corps of Engineers. The Regional Frequency program (14) was used by the Charleston District office to estimate missing records by correlation with nearby long term stations and to recompute the statistics. Data for these 21 stations are listed in Table 10.

Plots of mean and skew versus physical characteristics of the subbasins at the site of the stream flow gaging station were generated and are shown as Figures 15, 16 and 17. As can be seen, there exists little correlation between these physical parameters and their statistical counterparts.

b. Multiple Regression Analysis. Stepwise multiple regression analyses (15) were run on the corrected statistics to test for their possible relationships to the following basin characteristics:

1. drainage area,
2. main channel length,
3. main channel length to center of area,
4. mean slope, and
5. average elevation.

Various combinations of these parameters were also tested by regression analysis. This analysis indicated that most of the variation in the mean flow could be explained by the following relationship to the drainage area, A:

$$\log (Q_{50}) = C_M + 0.60 \log (A) \qquad (1)$$

where $\log (Q_{50})$ is the mean log of annual flood peaks, and C_M is a constant.

The regression parameter R^2 for this relationship was 0.80. Inclusion of the other parameters did not significantly increase R^2. Multiple regression analyses of the standard deviation, S, and skew, g, with respect to the basin characteristics resulted in low correlation.

c. Map Coefficients. Since no meaningful relationships could be found for the mean regression equation constant, C_M, the standard deviation, and the skew, they were plotted on regional maps (Figures 18, 19 and 20 respectively) to determine their geographical variation.

The regression equation and map coefficients were used to compute annual flood peak frequency curves in the Charleston area. Using Figures 18, 19 and 20 the values for C_M, standard deviation, and skew were found to be 1.8, 0.35 and -0.10 respectively. The drainage areas were used with C_M to calculate the log mean peak flow at several points in the Turkey Creek and Goose Creek basins from the following equation:

$$\log (Q_{50}) = 1.8 + 0.60 \log (A) \qquad (2)$$

The log of the annual flood peak with flow Q for a given exceedence frequency, P, was found from the following equation:

$$\log Q(P) = \log (Q_{50}) + K(P,g)S \qquad (3)$$

where K(P,g) is a function of exceedence frequency, P, and skew, g, obtained from tables derived for Pearson Type III coordinates (16). The annual flood peak frequency curves derived for points in the Turkey Creek and Goose Creek basins are plotted in Figure 21. Due to the large impoundment on Goose Creek little value can be put directly on the frequency curve for this basin. It is much more representative of the peak inflows to the reservoirs than the outflow.

15. RECONCILIATION OF DISCHARGE FREQUENCY CURVES. The regionalized frequency curves developed for Turkey Creek at the mouth, Turkey Creek at the SCLRR crossing, an unnamed tributary to Turkey Creek, and Goose Creek above the reservoir dam were compared to the points developed by the rainfall-runoff approach using HEC-1. For each of the three subbasins considered on Turkey Creek, the 100-year storm event is of approximately equal magnitude for each of the two methods. However, the regionalized frequency analysis produced lesser flows for the smaller exceedence intervals than did the rainfall-runoff method.

To reconcile differences in discharges predicted by the rainfall-runoff method and the regionalized frequency analysis it was first necessary to examine the inherent shortcomings of both methods. While the rainfall-runoff method produces site-specific discharge values that have been calibrated with an actual measured event, it cannot assess the flows that would occur for storms having return frequencies greater than 100 years. This limitation is directly linked to the nonavailability of rainfall information for high frequency events. Additionally, this method attaches a significant level of confidence in the values selected to reproduce an actual measured event. Truly there are many sets of parameters that would capture the calibration event. Just as assuredly, these different sets of parameters would yield different discharge values at either limit of the return frequency curve.

By definition a regionalized frequency analysis is one that is based on large areal considerations. Geographic homogeneity and consistent land use throughout the region being evaluated are implied. The best available data for use during this study were taken from stream flow gages that were located both in piedmont and coastal areas. The majority of these gages were located in watersheds that drained nonurban, open or agricultural land. By contrast, the Turkey Creek basin is moderately urbanized and is under tidal influence.

Since neither method proves its superiority over the other, four criteria were followed in establishing the return frequency curve that would be adopted as the curve upon which design and flood damage computations would be based. These criteria are presented without supporting arguments for their use and are:

1. Values of C_M and S selected for use in equations (1) and (3) of the preceeding paragraph should be bounded by the values shown on their respective regional maps;
2. The skew will be set equal to 0.0 to allow for a margin of safety;
3. The discharge value of the final adopted curve at the 100-year return frequency should approximate that calculated by the rainfall-runoff method; and
4. The adopted frequency curve should intersect the flood flow produced by the standard project storm between an exceedence interval of 200 to 1000 years.

The three statistical parameters used to compute the adopted frequency curve were adjusted to satisfy the above criteria at the mouth of Turkey Creek. These adjusted values were held constant and applied to correct the frequency curves at the mouth of the unnamed tributary, and at the SCLRR crossing of Turkey Creek. The resulting adopted frequency curves, along with the points determined by the rainfall-runoff method, are shown on Figures 8, 9 and 10. The deviation between the two methods at the low end of the frequency curve is attributable to the high degree of urbanization within the Turkey Creek basin which is not reflected by the regional frequency curves.

16. HEC-2 CALIBRATION. The computer program, HEC-2 (17), can compute the water surface profile for river channels for either subcritical or supercritical flow conditions and consider the effects of various hydraulic structures such as bridges, culverts and weirs. In order to effectively model the Goose and Turkey Creek reaches under consideration with this computer program so that profiles for various frequency floods can be determined, a procedure of model calibration was undertaken. This procedure entailed developing the model by using cross-sections and data to describe various bridge and culvert geometries. During calibration, modeling parameters, such as Manning's "n" were adjusted so that a simulated profile was representative of the observed water surface profile used for model calibration. The calibration storm chosen was June 1973 for which some highwater marks are available on Turkey Creek.

a. Goose Creek. Goose Creek was modeled from approximately one mile above its mouth to a point on the downstream side of the dam at Goose Creek Reservoir. Sixteen cross-sections were used to define the channel and overbanks, two of which describe the bridge geometries at the Rhett Avenue crossing. A diagram of the cross-section locations used in the model is shown in Figure 22.

No calibration data are available on Goose Creek; however, one water surface elevation on Turkey Creek near its mouth was used to calibrate the simulated water surface elevation in Goose Creek near the Turkey Creek confluence. The June 1973 calibration storm conditions for the backwater computations in Goose Creek were defined by the discharge from the reservoir, the discharge from Turkey Creek and the starting water surface elevation at the mouth of Goose Creek, which was set at 3 feet msl. The Goose Creek model calibration was considered acceptable with a simulated water surface elevation of 6.76 feet msl on Goose Creek just downstream from the Turkey Creek confluence. The observed water surface elevation at the mouth of Turkey Creek was 6.88 feet.

The initial values of Manning's "n" were selected using USGS Water Supply Paper 1849 *Roughness Characteristics of Natural Channels*(18), and from data received from the Charleston District office on Manning's coefficients in South Carolina. The final "n" values used during the Goose Creek calibration ranged in magnitude from 0.16 to 0.18 for overbank areas and from 0.07 to 0.08 for the channel. These values fall in the upper range of the normally acceptable values of "n" for this type of creek which meanders through swampy terrain.

The contraction and expansion coefficients used in computing transition losses were set at 0.3 and 0.5 respectively for the unbridged cross-sections in Goose Creek. For the cross sections defining a bridge opening where transitions are abrupt, a contraction coefficient of 0.06 and an expansion coefficient of 1.0 were used. These coefficients were selected from suggested values in the HEC-2 users manual for different types of transitions.

b. Turkey Creek. Turkey Creek was modeled from its mouth to a point 740 feet upstream from the Interstate 26 highway crossing. Six crossings were modeled and have been discussed in a previous section of this report (see paragraph 6).

The Turkey Creek model was developed from the surveyed cross-sections and special modeling data were developed from the bridge or culvert geometries at each of the crossings. A diagram of the cross-section locations used in the model is shown in Figure 23.

Calibration data for the June 1973 storm event consist of a few high water marks from the mouth to just upstream of the SCLRR crossing. The model was calibrated considering the observed highwater marks and also the probable profile conditions upstream where no calibration data are available. The upstream consideration was most important since an accurate downstream calibration may not necessarily produce reasonable conditions upstream, especially when there exist several crossings which greatly constrict the flow.

Figure 24 shows the water surface profile of the simulated event and also the observed high water marks for the June 1973 flood. The simulated profile compares satisfactorily to the observed elevations, and the upstream elevations are reasonable considering the constrictions due to the roadway crossing.

The flow conditions for the calibration run for Turkey Creek were completed by the rainfall-runoff model HEC-1. The starting water surface elevation imposed at the mouth was defined by the observed high water marks for the calibration storm.

The Manning's "n" selected during final model calibration varied along Turkey Creek, and was also specifically defined for the culverts in the system. For the natural river cross-sections from the mouth to the SCLRR crossing the "n" values assigned were 0.18 for the overbank areas and 0.06 for the channel. Just upstream from the railroad and through the trailer park the "n" values selected were 0.14 for the overbank areas and 0.06 for the channel. Values of 0.07 and 0.03 for the overbank areas and channel, respectively, were used for the remaining upstream reach of the Creek. The lower upstream "n" values were chosen to reflect the channel improvement which had been made above the railroad crossing in the past and the less dense undergrowth in the overbanks.

The contraction and expansion coefficients were 0.3 and 0.5 for the normal transitions between cross sections on Turkey Creek. They were increased to 0.6 and 1.0 for the abrupt transition at roadway or railroad crossings.

The Turkey Creek storage-outflow relationships developed for the computation of runoff by the unit hydrograph method were compared to those determined by the HEC-2 model. Where necessary, HEC-1 values were adjusted to agree with those of HEC-2.

17. *EXISTING AND FUTURE RUNOFF AS EFFECTED BY URBANIZATION.*

a. Effect of Existing Urbanization on Runoff. The peak discharge frequency curves for annual events, Figures 8, 9 and 10, were translated to a partial duration series by applying the Langbein criteria cited in *Statistical Methods in Hydrology* (16). Calculation of flood damages will be based on the stages produced by these translated discharges.

Justification for modification of the annual event curves is based on the present state of urbanization in the Turkey Creek watershed. The number of acres presently dedicated to the five land uses of residential, commercial, industrial, wetlands, and open/vacant were estimated. The estimated areas were multiplied by typical values for degree of imperviousness and a weighted average degree of imperviousness was calculated for the basin. Typical values for degree of imperviousness by land use, as determined by the consultant during previous studies, are shown as Table 11.

The average degree of imperviousness for the entire basin was calculated to be 36%. This value is several times greater than that for rural open or agricultural lands. In a previous section of this report the annual event discharge frequency curve was adjusted by applying statistical parameters that were developed from gaging station data largely located in nonurban areas. The use of the partial duration curve for low frequency return events will compensate for the effects on discharge of the urbanization present in Turkey Creek.

b. Effect of Future Urbanization on Runoff. As was mentioned in a previous section of this report, Turkey Creek watershed is situated in Berkeley, Charleston and Dorchester Counties, which are commonly referred to as the Trident area. Development plans have been formulated for three subsections of the Trident area that encompass the watershed. Data from two studies (19, 20) are readily available, while that from the third, the North Charleston Development Plan, are not available at this time, but shall be in the near future.

The comprehensive development plan for the City of Hannahan, South Carolina, encompasses the portion of the watershed between its mouth and the SCLRR crossing. Some 26% of the total land area of the basin lies within these boundaries. Growth, or urbanization, is not expected to occur in the watershed owing to the presence of man-made and natural limitations such as the safety restriction and noise exposure zones associated with approach/departure air lanes of the Charleston Air Force Base/Municipal Airport. Unstable soils, flood plain zoning, reserved land for the Goose Creek water treatment plant, and the nature of the present open marshy areas themselves all serve to make future urbanization impractical. For the most part, the present community environment will be maintained with the most significant changes expected to occur in improvement of recreational opportunities and the improvement of present residential areas through the piecemeal replacement of older homes.

The plan also recommends the extension of Redeemer Drive across Turkey Creek to provide better access for local area residents. It has been assumed that the sizing of the bridging structure will be compatible with the findings of this study as they relate to existing bridge structures.

The most upstream portion of the watershed, from the Charleston Municipal Airport to the divide, some 31% of the total land area of the basin, will be developed in accordance with the proposed master plan for the Charleston Air Force Base/Municipal Airport. Essentially, this plan recommends the municipal airport be greatly expanded by the year 1995. To this end, the Charleston County Aviation Authority has acquired an additional 1400 acres of land adjacent to the existing complex. The impact of this development on the Turkey Creek watershed will be negligible since the acquired land and proposed development lie within an area drained by the Ashley River. At most, this expansion will create an environment favorable to increased urbanization of that portion of the watershed not yet discussed.

This midsection of the watershed, that area between the SCLRR crossing and the Charleston Municipal Airport will be developed in accordance with the North Charleston City development plan, as yet unpublished. Development in this area will probably be spurred by the need for attendant commercial services which will be created by the expansion of the Charleston Municipal Airport.

This subarea contains some 43% of the total land drained by Turkey Creek and is already about 60% developed. The present degree of imperviousness for this subarea is calculated to be about 30%. Assuming the undeveloped land is utilized by future commercial-industrial activities and recalculating the subarea degree of imperviousness yields a value of about 40%. The effect of this change is minimal upon the weighted average value for the entire watershed.

The existing condition average weighted degree of imperviousness was calculated to be about 36% for the entire watershed. The future condition average weighted degree of imperviousness is about 38%. The change in degree of imperviousness between existing and future conditions is so slight that runoff values developed for existing conditions will also be used when evaluating designs under future conditions.

18. TIDAL EFFECTS. Tidal action in the downstream reach of Turkey Creek is pronounced. Tidal stages in Turkey Creek have a departure of 1.5 to 2.0 feet above normal tide elevations in Charleston Harbor and a lag time of approximately 3 to 4 hours. Normal tides in Charleston Harbor range between ±2.6 feet msl with spring tides achieving an elevation of about 4.2 feet msl. The Corps of Engineers, Charleston District, has investigated tidal fluctuations in the vicinity of Charleston Harbor and has computed tidal surge heights associated with hurricanes of various return frequencies. Figure 25, a tidal surge frequency curve, is a graphical representation of the findings of their study.

19. PROJECTED FLOOD DAMAGES

a. General. Damages to structures within the Turkey Creek flood plain are attributable to flood stages created by rainfall-runoff, hurricane tidal surges and combinations of both. The expected annual damages from storm runoff and pure tidal surge can be readily computed. It would be difficult to ascertain the expected annual damages caused by combinations of these events. The difficulty lies in the development of a joint probability distribution for the coincident frequency of fluvial and tidal events. If both events were completely independent, the probability of simultaneous occurrence of both would be equal to the product of their respective probabilities. It is known that they are not independent. Hurricanes are generally accompanied by rainfall, but the converse is not always true. The degree to which these two events correlate with one another presents a second encumbrance to formulation of a coincident frequency analysis. Synthetic hurricanes of various frequencies are often times generated without regard to rainfall, whereas rainfall frequencies, as developed in the Weather Bureau's *Rainfall Frequency Atlas*, are based on data from all classes of storms including hurricanes. Without first eliminating non-hurricane associated rainfalls from the Atlas and correlating the remaining rainfalls with the data used in developing the criteria for synthetic storm construction, the joint probabilities cannot be quantitatively determined. Since such an undertaking is beyond the scope of this study, it appears logical to assume that a reasonably liberal estimate of the damages produced by rainfall runoff, hurricane tidal surges, and combinations of both can be achieved by summing the damages attributable solely to storm runoff and those solely attributable to tidal surges. Therefore, two sets of flood damage computations were performed; one set for runoff events, and the other for tidal surge events.

The procedure for assessing damages from both types of events was similar. All structures in the flood plain were given a numerical designation and first floor elevations and present market worth were determined for each by field survey. The percentage of damage to structures and contents was related to the depth of flooding by using Table 12. The value of commercial and business inventories and physical plant values were determined by directly interviewing personnel engaged in the management of these activities. These data for commercial, business and public properties are shown as Table 13.

b. Economic Reach Descriptions. To facilitate the calculation of flood damages, the watershed was subdivided into three reaches of nearly homogeneous land use.

Reach 1 begins at the mouth of Turkey Creek and terminates at the SCLRR crossing. The majority of structures in this reach are single-family residential homes. There are approximately 310 structures in this reach that have a first floor elevation of 20.0 feet msl or less. Homes in this reach are most susceptible to the backwater effect of tidal surges. The average 1975 estimated market value of homes in the southern half of the watershed is $37,000. Homes in the northern half have been valued at $28,000. These values were obtained from a local area realtor who visually inspected the homes in the basin. He reported so little variation exists in the market value of individual homes that it would be impractical to record a unique value for each. There are no commercial establishments in the flood plain in this reach and public structures are limited to two sewer lift stations.

The next upstream reach, Reach 2, begins at the SCLRR crossing and extends upstream to the U.S. Highway 52 crossing. Again, structures are predominantly residential, but there is some mix with commercial establishments. In this reach there are approximately 140 mobile homes with a first floor elevation of 20.0 feet msl or less. Their average market value has been estimated to be $6000 each. Some mobile homes had greater value and some less. These variations are reflected in the tabulation of flood damages, but for the sake of brevity are not reported herein. Additionally, some 15 commercial establishments are located in the upstream portion of this reach. Flood stages in this reach are largely a result of runoff which ponds at the entrance to the culverts beneath the SCLRR embankment.

Reach 3 begins at the U.S. Highway 52 crossing and terminates at the upstream study boundary. This is the least densely populated reach. There are no residental structures and only a few commercial activities within this area. There is no historic basis for assessing the nature of flood damages for this reach since none have been reported.

Aerial photographs of the Turkey Creek flood plain are contained in Appendix A to this report. The numerical designation assigned to each structure, the field survey base line and location of cross-sections are shown on these photographs. A table of structure numbers and first floor elevations are also contained in this Appendix.

c. Fluvial Flood Damages. An assessment of damages produced from storm runoff began with backwater calculations for each reach of Turkey Creek for the 2-, 10-, 25-, 50-, 100-year return frequency storms and the standard project storm. These calculations were performed using the HEC-2 computer model. Input requirements included the appropriate reach discharge (selected from Figure 8, 9 or 10), the corresponding discharge on Goose Creek and a starting water surface elevation at the mouth of Goose Creek at the Cooper River. The water surface elevation of mean high tide was used as a starting condition for all backwater calculations. The backwater elevations output by the HEC-2 model were then plotted as a function of the discharges that produced them. The resulting three stage-discharge curves are shown as Figures 26, 27 and 28.

The flood stages from the selected return frequency storms and the standard project storm were then utilized to compute damages to residential structures in Reaches 1 and 2. The method of computation is best explained by an illustrative example. This example explains how residential flood damages were determined for the 100-year return frequency storm. This storm produced an average flood stage of about 9.2 feet msl over Reach 1. Two feet were subtracted from each first floor elevation to determine the approximate ground elevation for each residence. The ground elevations corresponding to each residence in Reach 1 were compared to the flood stage. Structures with ground elevations greater than 7.2 feet, first floor elevations of 9.2 feet, were assumed to be undamaged by the flood. Structures with lesser ground elevations were assumed to be damaged and were recorded by structure number. The extent of damage, in percent of the structure's value, was determined by calculating the depth of water above ground elevation and then employing Table 13. The actual cost of flood damage, in dollars, was determined by multiplying the structure's market value by the percent damage it sustained. The depth of flooding, percent damage and dollar value of damages was recorded for each structure. The damage, in dollars, to each structure was summed to yield a total of $240,000 in residential flood damage for Reach 1 for the 100-year fluvial flood stage. This process was repeated for different stages caused by the other return frequency and standard project storms. In this manner, sufficient information was generated to produce flood stage-damage relationships for each classification of residential, commercial and public property. The data is plotted and shown on Figure 29. A detailed drawing is also given on the figure to show the stage-dollar damage relationship for the first 10 feet of flood stage. Figures 30 and 31, the flood stage-dollar damage curves for Reaches 2 and 3 were developed through a similar process.

d. Tidal Flooding. The damages from hurricane tidal surges were determined by selecting stages from Figure 25, Tidal Surge Frequency Curve, at the 2-, 10-, 25-, 50-, and 100-year return frequencies. These stages were then transferred to the Turkey Creek basin without attenuation and without the occurrence of any rainfall. Damages by property classification, by reach, were estimated by reading the dollar damage from the appropriate stage-damage curve (Figure 29, 30 or 31) at the corresponding surge elevation. The calculation of damages produced by the standard project hurricane was more tedious in that it produced the highest flood stage, and as a result, damages had to be individually tallied by structure and summed in the manner described in the previous section.

e. Average Annual Damages. The damages from several frequency floods were calculated and weighed by the probability of their occurrence to obtain average annual flood damages. Damage calculations were based on existing structures. Further, the average annual damage has been based on events having a return period frequency of 200 years or less.

Separate calculations have been performed for flooding produced by rainfall-runoff and that produced by hurricane tidal surges. Calculations were performed by nature of flood, by reach, and by property classification. These calculations are shown on Tables 14 to 23. Table 24 summarizes the average annual damage by reach and land use classification. Damages produced by the standard project storm and the standard project hurricane are also shown on Table 24.

The sum of the average annual damages, $47,069, is taken as the best estimate of the most probable average annual flood damage produced by the occurrence of rainfall and/or hurricane tidal events.

20. PRELIMINARY DESIGN PROJECT ALTERNATIVES

a. General. The project alternatives studied for Turkey Creek were preliminarily evaluated in accordance with criteria contained in applicable engineering manuals and those contained in the scope of work which is basin specific for Turkey Creek. Because this is a highly urban area, high degrees of protection should be considered.

The most desirable protection for this area would be protection from the standard project flood. From the Hydrology Report, the standard project fluvial flood produces a starting water surface elevation at the confluence of Turkey and Goose Creeks of approximately 9.8 feet msl. This elevation is two feet above the roadway elevation at the Murray Avenue bridge and the resulting upstream backwater, computed with the use of HEC-2, overtops Highway 52 and the J.M. Field access road crossing. The standard project hurricane produces a water surface elevation of seventeen feet msl at the mouth of Turkey Creek. This elevation would almost inundate the entire flood plain. The high cost of protecting against this tidal surge and/or the standard project fluvial flood does not appear to be within the allowable costs for improvements. From the criteria stipulated in the work scope it is desirable to at least provide protection against the 100-year storm and thus the combination of the 100-year fluvial storm and the 100-year tidal surge was used as an initial screening device for alternative evaluation. Several of the alternatives are also further screened in this project using only the 100-year fluvial conditions.

Under existing conditions the 100-year fluvial flood overtops Murray Avenue and Highway 52 which are designated as major thoroughfares. These crossings are not overtopped by the 50-year fluvial flood and thus they meet bridge design criteria under present conditions for passing the 50-year frequency flood without overtopping.

A combination of the 100-year tidal surge and 100-year fluvial flood overtops both major crossings. The major cause of the overtopping is due to the high tide conditions at the mouth of Turkey Creek. The 100-year fluvial flooding causes damage to greater than 31% of the residences located in the flood plain downstream from the SCLRR crossing, while the 100-year tidal surge causes damage to approximately 50% of these residences. The number of structures damaged resulting from each of these two conditions is similar in the flood plain above the SCLRR crossing.

Each of the alternatives listed below are described in the following paragraphs as to their specific function, hydraulic design, and its resultant effect on the water surface elevation. A short summary of the resulting degree of flood protection in the basin is given. For non-hydraulic designs a discussion of their function, application and potential use for flood protection in the basin is given.

Preliminary Design Project Alternatives:

1. Remove silt and debris from blocked SCLRR culvert;
2. Channel improvement from Murray Avenue to the SCLRR, with railroad culverts fully open;
3. Levees on each side of Turkey Creek with local drainage from the mouth to Highway 52, with railroad culverts fully open;
4. Levees, channel improvement, and railroad culvert fully open (combination of alternatives 1, 2 and 3);
5. Transverse dike at mouth of Turkey Creek, with fixed gate, with tidal gate, with tidal gate and pumps;
6. Relocation of structures subject to damage, flood proofing existing structures, or raising floor elevations of existing structures and possible combination of all three;
7. Maintain present conditions.

b. *Alternative 1: Remove Silt from SCLRR Culvert.* The present condition of the SCLRR culvert shows that siltation has reduced the effective area of the culvert opening by one-third. This condition causes excessive backwater on the upstream side of the railroad crossing. Using HEC-2 the culvert geometry was set to represent the fully opened conditions and a reduction in the backwater resulted. Table 25 shows the average water surface elevations for the three major reaches for Alternative 1. Figure 32 shows the water surface profile of Alternative 1. The water surface elevations can be compared with the existing conditions whose water surface elevations are given in the table and the figure. More specifically, the water surface elevation immediately upstream of the railroad crossing (cross section 8) was reduced by more than two feet using the total capacity of the culverts. This decrease of backwater reduces the residential and commercial damages of the structures between the railroad and Highway 52 for the 100-year storm by 90 percent.

Any further reduction of backwater and subsequent damages by modification of the railroad crossing (i.e., an additional culvert through the railroad embankment) would not be cost-effective for Reach 2. Local engineers have reported extreme problems and high costs for the initial installation of the second culvert. A more feasible alternative may be to relocate the trailers still subject to damage after the culvert has been fully open. A problem with this approach is the fact that there is no relocatable space in the existing trailer park. A new location would have to be found with more costly site preparation than required for a move within the park itself.

The removal of the sand and silt from the railroad culvert to allow the passage of water through the design opening area is presently considered cost-effective. Therefore, this alternative has been considered as a minimal condition which will be completed and thus it is included for all remaining alternative plans.

c. Alternative 2: Channel Improvement from Murray Avenue to the SCLRR with Railroad Culvert Fully Open. The major problem in Reach 1 for the 100-year storm is the high initial water surface elevation at the mouth due to conditions in Goose Creek. The actual rise in water surface elevation for the 100-year flow from the mouth to the SCLRR crossing is only 0.6 of a foot. Therefore, any channel modification can only act to reduce this difference since the water surface elevation at the mouth of Turkey Creek is determined by the backwater in Goose Creek. Unfortunately, this severely limits the usefulness of channel modification in Reach 1.

Some attempts at reducing this backwater effect by enlarging the channel were made. Channel improvement consisted of channel modification due to excavation and channel straightening of meandering reaches from Murray Avenue (cross-section 14) to the SCLRR crossing (cross-section 8.9). Some improvement of the Turkey Creek channel has been already accomplished. During a recent on-site inspection, it was noted that the portion of the creek between Murray Avenue and the SCLRR, Reach 1, differed greatly from the cross-sections that were surveyed during May 1975. Upon comparing plots of the cross-sections with photographs taken during and after the flood of June 1973, it was observed that channel enlargement and straightening had occurred in the interim. Therefore, these channel modifications were performed in two phases. The first phase was completed after the June 1973 flood, but prior to the survey of May 1975. The second phase was undertaken after the survey and completed during August 1975. As this Reach exists now, it is of fairly uniform top width of about 30 feet, with a parabolic cross-section that is about 3 feet deep at its center. The material dredged from the creek has been placed about 15 feet from the right side of the channel looking downstream in a more or less continuous levee that stands about 10 feet high. The slope of the levee appears to be at about the angle of repose for the excavated material, clayey sand. This levee begins near the SCLRR culvert outlet and extends to a point some 200 yards downstream of Rembert Drive. The location of this most recent channelization is depicted by the cross-hatched area shown on Figure 2. Turkey Creek appears on this figure much like it existed during the flood of June 1973. It should be noted that the channel approach between Murray Avenue bridge and the end of the cross-hatched area was enlarged and straightened by the modifications made under the first phase of work which was completed during September 1973. The channel at the Murray Avenue bridge is now nearly as wide as the bridge crossing itself.

Both phases of work were performed by the Hannahan Public Service Commission to specifications outlined by the U.S. Soil Conservation Service. In comparing the existing channel to its specifications, it appears that either the work was done by a skilled dragline operator without the benefit of a surveyor, or that such work was performed to specification and subsequent weathering and siltation has caused alterations.

An optimization of channel size was studied by adjusting the bottom width and permissible side slopes to produce the minimum backwater effects in the reach. The channel sizes were limited since it was found that extensive channelization would only cause slight additional decreases in the water surface elevation. The channel cross-sections were modified and the computer model HEC-2 was used to predict the water surface elevations produced by the 100-year flood event.

The elevations of the channel invert, defined by each cross-section, were reduced throughout the stream stretch under consideration to produce a channel slope similar to the natural channel slope between cross-sections 14 and 13. Figure 33 shows the natural channel invert defined by each cross-section and the channel invert as a result of channel improvement for this alternative. Throughout the stream stretch a bottom width of 50 feet and side slopes of one vertical to three horizontal were used. The Murray Avenue bridge opening was not modified with channelization since the backwater produced by the constriction was minimal (0.05-0.15 feet) for all flood frequencies studied, and the roadway was not overtopped by the 50-year frequency storm.

The resulting average water surface elevations for the three reaches are shown in Table 25, and the water surface profile is given in Figure 32. Under existing conditions the total backwater from the mouth of Turkey Creek to the SCLRR produced an increase in water surface elevation of 0.6 of a foot. With the channel modification described in this alternative, the increase between the same two cross-sections was 0.20 of a foot (see Figure 32). The channel modification in Reach 1 also reduced the water surface elevation in Reach 2 and at the first cross-seciton in Reach 3.

The maximum allowable preliminary channel design velocity is that velocity where erosion commences. This is considered as 3.5 feet per second, and it was not exceeded where the channel was modified. The maximum channel velocity obtained in Reach 1 for the 100-year storm was approximately 1.5 feet per second. The velocity through the Murray Avenue opening is less than the 5 feet per second criteria for placement of revetments.

The 100-year storm causes extensive damage in Reach 1 and although the reduction in average water surface elevation for Reach 1 is only 0.2 of a foot as a result of channel improvement, the dollar damage is reduced by 20 percent. The average elevation for Reach 2 as a result of channel improvement and the railroad culvert fully open is decreased an additional 0.5 of a foot from Alternative 1. The water surface elevation of 11.1 feet msl causes minimal damage in Reach 2 since the zero damage stage is at 11.0 feet msl. The average water surface elevation as a result of channel improvement is reduced in Reach 3; however, this reduction is only significant between Highway 52 and the J.M. Fields access road. Therefore no reduction in damages for Reach 3 is accomplished by channel improvements in Reach 1.

d. Alternative 3: Levees from Mouth of Turkey Creek to Highway 52 (Reaches 1 and 2) with Railroad Culvert Fully Open. Levees in urban areas are constructed to protect residential, commercial and industrial developments against inundation resulting from high stages in a river or tidal area. It is also necessary to provide leveed areas with local drainage. Interior drainage systems may consist of various combinations of ditches, conduits, gravity outlets, ponding areas and pumping stations.

With the use of HEC-2 the water surface profile produced by the 100-year storm was simulated with preliminary design levees on each side of Turkey Creek between the mouth and Highway 52. The levees were located on the left and right banks of the channel. The banks, as defined by the model, were developed from cross-section data. These levees restricted the flow to the channel and produced the average water surface elevations for the three major reaches as shown in Table 25 and the water surface profile as shown in Figure 34. Final levee design which is to include three feet of freeboard with provision for local drainage, will protect the leveed area 100 percent against the 100-year fluvial storm. Levees in Turkey Creek have the additional benefit of providing protection against tidal surges.

The protection described here relates only to the protection afforded the area from the high stages in the Creek. Providing for local drainage in the Turkey Creek basin is expected to be a major undertaking with resulting high costs. The basin has relatively small side slopes and is highly urbanized, therefore the space that could be allocated as non-drainage ponding areas is very limited. The discharge capacity of gravity outlets would soon be exceeded as the river stage rises and as the interior drainage flows reach the line of protection. It is probably that pumping stations would have to be used to discharge interior drainage flows over the levees when free outflow from gravity outlets is prevented by high outfall stages. This problem is complicated by the large number of small local inflow locations. There does not appear to be one or two major local inflows that account for the majority of the drainage. This creates a costly problem when considering interior flood relief and is a typical symptom of flooding problems in small highly urbanized areas.

The channel velocities through the leveed channel for the 100-year storm are generally less than 2 feet per second, and never greater than the preliminary design criteria of 3.5 feet per second. The average water surface elevation for Reach 3 is not increased above existing conditions as a result of the downstream levees.

One major concern in addition to local drainage in the application of levees to Turkey Creek is the fact that the Murray Avenue bridge is overtopped by the 100-year storm. For levees to be constructed with 3 feet of freeboard on each side of Turkey Creek, special consideration must be given to the Murray Avenue crossing. Levee design to allow the normal flow of traffic during low flows and to protect the area at the bridge crossing during flows which rise out of the normal banks must be studied.

The preliminary levee design with 3 feet of freeboard for the 100-year storm would also protect Reach 1 against the standard project fluvial flood, but would not protect Reach 2.

e. Alternative 4: Levees, Channel Improvement, Railroad Culvert Fully Open (Alternative 1, 2 and 3 combined). Alternative 4 which is a combination of the first three alternatives was studied to determine the effect of channel improvement on the levee height necessary to protect the leveed area against the 100-year storm. The average water surface elevations for the three major reaches are shown in Table 25, and the profile is shown in Figure 34. Using HEC-2 the levees were located at the banks of the new channel resulting from channel modification.

From Table 25 it can be seen that the levee height for Reach 1 can be reduced by one foot; and for Reach 2, by approximately 2 feet to provide the same protection as given by Alternative 3. Thus, the choice of combination of levee height and channel capacity improvement to provide protection would be chosen based on economic considerations which will largely be governed by the suitability of the dredged channel materials for use in levee construction.

f. Alternative 5: Construct Transverse Dike at Mouth of Turkey Creek.

1. General. The fifth alternative method of providing flood protection to Turkey Creek area residents involved the evaluation of a dike constructed transverse to the flow direction of the Creek at its mouth. This dike would bar entry of tidal surges into the Turkey Creek basin and would thus eliminate practically all flood damages produced by hurricane tidal surges. This alternative proves to be a more viable solution in that for the same return period frequency event the hurricane tidal surge invariably produces significantly higher flood stages than does the fluvial flooding caused by the corresponding rainfall event.

2. Dike Geometry. Two primary considerations govern the sizing of the tidal dike. First, it should be high enough to prevent flooding from the standard project hurricane (SPH), and secondly, in the absence of a tidal surge, it should pass the fluvial floods without creating detrimental backwater. The elevation of tidal stage produced by the SPH is approximately 17.0 feet at the mouth of Turkey Creek. The top elevation of the dike was established at 20.0 feet by adding three feet of freeboard to the maximum tidal surge to allow for a margin of safety. The net opening area of the gates was determined by providing approximately the same cross-sectional area as exists at the Murray Avenue crossing. Dike width at the base is predicated upon local soil characteristics which should be stable at a side slope of 1 horizontal to about 2.75 vertical. The geometry of the proposed dike is shown in Figure 35. These findings are in agreement with those reported in the reconnaissance report (1).

3. Fixed-Gate and Dike. By definition, a fixed-gate dike is a dike that has its opening temporarily sealed throughout the duration of a flood-producing event. This would entail the placement of sheet metal or logs or some other such blockage material at the beginning of a storm, or surge, that would stay in place until the storm or surge subsided and then it would be removed to permit drainage of the interior of the basin. To evaluate the flood stages produced under these conditions, a stage-storage curve for the entire basin was first developed. The curve shown on Figure 36 represents the basin-wide available storage for a given stage and was determined by summing the appropriate values which were output by the HEC-2 model. Calculations have shown that about 87% of all available basin storage below 20.0 feet msl could be accounted for in Reach 1.

The required storage for the preselected rainfall frequency events was then determined by multiplying the basin area times the excess rainfall occurring during duration of the storm. The appropriate conversion factor was then applied to convert this storage to storage required in acre-feet. The point values of required storage for each event have been superimposed upon the basin-wide available storage curve and are shown on Figure 37. Also shown on this figure are the deviations the preselected return period tidal surges would reach in the watershed if there were no dike. It can be seen from an examination of Figure 37 that if the excess rainfall from a given return period event was completely stored behind the dike it would produce a flood stage significantly higher than the corresponding return period hurricane tidal surge without a dike. Under these conditions local area residents would only receive protection when tidal surges occurred in the absence of a significant rainfall. At all other times flood stages would be heightened and negative protection would result. Accordingly, this alternative has been evaluated as being undesirable.

4. Dike and Tidal Gate. A tide gate is a simple mechanical device activated by a change in tidal elevation. If the head on the outside (tidal side) is greater than that on the inside (fluvial side), the gate will close. If the reverse is true, the gate will open. To effectively assess the feasibility of reducing flood damages through the use of a dike with tide gates, it first was necessary to examine the time variance of fluvial and tidal stages independent of one another and then under conditions of dependence. The time-varying stage of the 100-year hurricane surge was provided by the Charleston District of the Corps of Engineers and is shown as Figure 38. The 100-year outflow fluvial stage for the most downstream section of Turkey Creek was developed from both HEC-1 and HEC-2 model outputs and is shown as Figure 39. Likewise, a 100-year inflow hydrograph for the total basin was developed and is shown as Figure 40. These three figures depict the time variance of fluvial and tidal stages independent of one another. Before the effects of their interaction were evaluated, a rating curve for the dike opening was determined.

The *Virginia Drainage Manual* (8) was used in establishing the rating curve for the culverts shown by Figure 35. The rating curve was determined independent of the elevation of head and tail waters. It was based on the head difference between the elevations of head and tail waters. This discharge-rating curve is shown as Figure 41.

The intensity and duration of hurricane rainfall is highly dependent upon the orientation and track of the hurricane as it approaches landfall. Undoubtedly, there is great variability in the paths individual hurricanes may follow. With respect to time, there also would be an infinite number of ways the peak runoff could occur relative to the peak tidal surge. Rainfall-runoff could peak before or after the peak surge occurred, or at any time in between. The worst possible combination that could occur would be for both events to peak simultaneously. This would create high elevations on the tidal side of the dike gate which would cause it to close. The closing of the gate would, in turn, cause greater ponding depths on the inside of the dike, thus producing higher flood stages and greater damages. This is the condition of dependence examined by this study.

The 100-year fluvial stage hydrograph is superimposed upon the 100-year tidal stage hydrograph so that their peaks occur simultaneously. The gate is assumed to be open and discharging until that point in time on the hydrographs where their initial intersection occurs. All discharge prior to this time is assumed to occur as though natural conditions existed. Discharges after this time were based on a routing of the total basin inflow hydrograph. Hand calculations were performed by determining the initial volume of water in storage at the time the gate closed. A small time step was selected, and the increase in tidal elevation during this time was read from Figure 38. The inflow at the end of this period was also calculated and converted to a stage assuming no discharge occurred. If the resulting fluvial stage was higher than the tidal stage, a discharge was computed using the culvert rating curve and the average value of the difference in head between tidal and fluvial stages during this time period. The volume of water discharged was then computed and subtracted from the initial storage volume. If the fluvial stage at the end of the time period was less than the tidal stage, no discharge was assumed to occur and the volume of inflow was added to the initial storage volume. The calculation of inflow volume; the comparison of fluvial and tidal stages; the calculation of discharge, if any; and the adjustment of fluvial waters in storage were iteratively performed for sequential time steps until the total basin inflow hydrograph had been routed.

As would be suspected, the 100-year fluvial discharge occurring under these conditions produced higher flood stages than would have been caused by the 100-year tidal surge in the absence of a transverse dike. Figure 42 is a plot of the stages resulting from the total basin inflow hydrograph routed through the dike culverts. Stages for the 100-year tidal surge and fluvial outflow for existing conditions are also shown for comparison.

Upon examining Figure 42, it is concluded that flood protection is only provided by the dike and tidal gates when hurricane tide surges are accompanied by sparse rainfalls, or none at all.

5. Dike, Tidal Gate and Pump. From the preceding subparagraph it can be seen that the construction of the dike interferes with interior drainage of the Turkey Creek watershed and that satisfactory discharge cannot be attained with a gravity outlet, irrespective of size. It is necessary to augment natural gravity discharge with a pumping station situated near the dike. It is envisioned that this additional discharge will be pumped over the dike and the effect of the additional flow on the tidal side will be negligible because of the large cross-sectional area available for storage.

With a pump, discharge from the interior will be possible even if the rise in tide level closes the gate and prevents gravity flow.

Two conditions were investigated. The first was a static condition where there was no flow through the culverts. A mass runoff curve was prepared for the 100-year frequency rainfall in accordance with the guidance given by EM 1110-2-1410 (21). Three pump sizes of 500, 750 and 1000 cfs were selected and allowed to pump at a uniform rate during the duration of the storm. The maximum storage required for these three pumps are 520, 194 and 15 acre-feet, respectively, which translate to corresponding flood stages of 9.6, 7.3 and 0.0 feet msl. Were the static condition to occur, a pump with a capacity between 750 and 1000 cfs would be required to keep the flood stage below 3.8 feet, the level of zero damage.

Since the static ponding condition is more a theoretical consideration than a natural reality, a second more probable condition of discharge by pump and gravity was investigated. The procedure for routing the flood was essentially the same as that previously described in subparagraph f (4). except that a constant discharge equal to the pump capacity being investigated was allowed to occur in each time step before any discharge was computed for the culverts. Close attention was paid to pump draw down so that its effect on the relative head difference between the inside and outside of the dike could be assessed. Appropriate consideration was then given when computing average head differences for flow through the culverts.

Two pumping schedules were used. The first allowed the pump to operate at a uniform rate beginning in time when the tidal gate closed due to the action of the surge and ending when the surge receded enough to allow the culverts to carry the discharge necessary to drain the interior.

Figure 43 depicts the resulting stage from operating a pump with a hydraulic capacity of 750 cfs under these conditions. Upon comparing the outflow stages for the dike with tidal gates without and with a pump, it can be seen that the latter condition reduces the maximum elevation of fluvial ponding to an elevation of 9.6 feet. While this elevation exceeds the elevation of zero damage, it represents a significant reduction in flood stage which attained an elevation of 11.4 feet for the dike with tidal gate and no pump.

The second pumping schedule began operating the 750 cfs pump some ten hours before the tidal gate would have closed by natural action of the tidal surge, or ten hours after the beginning of rainfall. By beginning the pump at this time, the tidal gate closed sooner and the pump was able to draw down the mean high water pond before the inflow from the total basin hydrograph began to exceed the rate of pumping. By reducing the amount of water in storage at the beginning of the storm, additional storage was created which was utilized as the inflow hydrograph neared its peak. The net effect of operating the pump under this schedule was to decrease the maximum fluvial ponding to an elevation of 7.4 feet. Figure 44 is the outflow stage that would result from operating the 750 cfs pump with this schedule. Pumping operations terminated when the culvert was able to pass a discharge sufficient to drain the basin. With respect to time, pumping terminated about 18 hours after it began.

Two larger capacity pumps of 850 and 1000 cfs were operated with the same pumping regime. These pumps produced maximum fluvial ponding stages of 6.2 and 3.5 feet, respectively.

From the foregoing, it can be seen that to provide complete flood protection from the simultaneous occurrence of the 100-year fluvial flood and tidal stage a transverse dike to bar entry of the tidal surge would have to be constructed. This dike would require tidal activated gates to maintain the existing environment of the watershed and to allow for release of waters under nonflood producing conditions. Additionally, a high capacity pump, 1000 cfs, would have to be provided to prevent the fluvial runoff from ponding above the elevation of zero damage when Turkey Creek was susceptible to the backwater effects of the 100-year tidal surge.

6. Preliminary Construction Costs. Initial estimates of the cost of providing protection against the simultaneous occurrence of the 100-year rainfall and the 100-year tidal surge were made to see if it is economically feasible to provide local area residents with this degree of protection. From the computation of flood damages, a value of $47,069 is expected in average annual flood damages. If the dike, gate and pumping station are assumed to have a service life of 50 years and money is available at 5-7/8% interest, a first cost of about $756,000 represents the prospective savings that would be attained if flood damages were reduced to zero damage elevation.

A preliminary estimate of the first cost to both construct the dike and culverts, and provide attendant engineering, design, and administrative services approximates $495,000 dollars. This value was determined by extracting the appropriate data from the reconnaissance report (1) and should be viewed as being conservatively low as the costs have not been adjusted for the inflation that has occurred between the years 1972-1975. An additional cost would be borne by the requirement for an increased dike size. The findings of the reconnaissance report indicated a dike 18 feet high was required while the findings of this study indicate a dike 20 feet high is required to provide the same protection.

Subtracting the costs of the dike and culverts, $495,000, from the total justifiable first cost of $756,000 allows $261,000 to be spent in making channel improvements and constructing the pumping station.

First approximations of pump station costs have been obtained from Plate 1 of EM 1110-2-3101 (22). Utilizing the least expensive curve yields costs in 1962 dollars of 290,000, 360,000 and 460,000 for station capacities of 500, 750 and 1000 cfs, respectively. Even if no channel improvements were made, these costs indicate that complete protection cannot be had for the simultaneous occurrence of the 100-year tidal and rainfall events.

g. Alternative 6: Relocation of Structures Subject to Damage, Flood Proofing Existing Structures, or Raising Floor Elevations of Existing Structures and a Combination of All Three. The Turkey Creek basin is a highly urbanized area with numerous single family dwellings. Reach 1 alone has 310 houses whose first floor elevations are less than 20 feet msl, 50 percent of which are damaged by the 100-year tidal surge. Reach 2 is mainly residential and includes 140 mobile homes and 15 commercial structures whose first floor elevations are less than 20 feet msl. Reach 3 has no residential structures and a few commercial establishments which would be susceptible to damage during a major flood.

The numerous structures which sustain some type of damage during storms of great magnitude make it uneconomical to relocate, flood proof or raise floor elevations of these existing structures.

h. Summary of Preliminary Alternatives. The ensuing discussion capsulizes the findings of the assessment of the six preliminary alternatives for control of flooding along Turkey Creek. It is unfortunate that problem solution must occur after the fact. A large number of structures have been built within the 100-year flood plain of this stream. Any proposed solution to provide relief from flood waters must be expensive as a consequence. The major finding from the evaluation of flood control alternatives is that the cost of providing the desired level of protection from the 100-year storm may not be justified from a consideration of economic costs and benefits alone, and that as a result, the degree of protection will possibly have to be downgraded to a less desired level.

During this phase of study, alternative project designs have been screened with respect to their effect upon the flow regime of Turkey Creek. During the next phase, each viable alternative will be evaluated with respect to its environmental and social effects. Additionally, the economic analysis of project alternatives will become much more refined during this second phase. All indicators now point to the economic costs being the major overall design constraint to the project. In all probability, final project design and level of protection afforded will be dictated by these economic constraints.

During the preliminary analysis many alternatives were found to be untenable because first approximations of their economic costs exceeded the benefits they created. These alternatives include flood-proofing of structures, raising floor elevations, relocation of structures and constructing a levee along the banks of Turkey Creek. Of these, the latter is the most defensible in that the initial estimated cost of the levee that would provide protection from all events up to and including the standard project hurricane is equal to approximately half of the benefits it would create.

However, the ill-defined natural drainage system for such a highly urbanized area would make the cost of providing interior drainage prohibitive.

Among the most favorable alternatives are restoring the hydraulic capacity of the blocked SCLRR culvert, channelization, and construction of a transverse dike with gates and pumps. Each of these alternatives produces a measurable benefit, but applied in combination, they create a synergistic effect. The first has the greatest single effect, while the third the most overall flood relief promise. The second of these alternatives proves to be the least sensitive. During the hydraulic analysis, it was found that flood stages on Turkey Creek were more a function of the flood stage on Goose Creek than a function of channel conveyance or capacity. Additionally, the effect of an improved channel is somewhat negated by the ponding effect of tidal action. The most benefit would be derived from an improved channel when the effects of these conditions were not present. They can be eliminated with the construction of a barrier dike at the mouth of Turkey Creek oriented transverse to its direction of flow.

Several different modes of discharging through the barrier dike were investigated. These included a fixed gate, a tidal gate, and a tidal gate augmented with a high capacity pump. During all modes of operation, the barrier dike was effective in removing all threats of damage from tidal surges. Damages from these surges approximated 70% of the total expected average annual damage.

In the absence of a tidal surge, the improved channel and additional discharge through the unblocked SCLRR culvert would greatly reduce flood stages on Turkey Creek. No significant increase in backwater would occur in the vicinity of the barrier dike since the gravity outlets will have been designed to pass natural and flood flows with only a slight increase in head. To illustrate, the 100-year fluvial flood can be discharged by these outlets with less than 0.3 of a foot difference in head. The mode of discharge becomes critical when hurricane tidal surges and rainfall events of the same return period frequency occur simultaneously. Under these conditions, a gate that was fixed in place at the start of the storm and not removed until it had subsided would cause fluvial flood stages to be greater than the tidal surge it was protecting against. Obviously, an undesirable effect. A dike with a tidal gate operating under these conditions would afford a limited degree of protection. During the analysis of the mutual 100-year events, it was found that flood stages on the inward side of the dike on Turkey Creek would only be 20% less than the stage produced by the tidal surge and would still cause catastrophic damages.

Of all the alternatives considered, the transverse dike with tidal gates and augmenting pump station is the most favorable. The barrier dike itself protects against damage from tidal surges. The tidal gates would provide natural drainage during non-flood periods and would serve to preserve the present ecology of the basin, if properly designed. During periods of flooding the pump station could be operated to keep flood stages below the elevation of zero damage. This would include operating the pump when flood stages were high on Goose Creek both from tidal surges and from runoff caused by rain storms.

Were economic cost no constraint and project design to be based solely on hydraulic considerations, the following improvements would be recommended:

- Construction of a transverse barrier dike at the mouth of Turkey Creek of sufficient height to bar entry of the standard project hurricane tidal surge.
- Installation of tidal activated gates at the dike site. These gates should have a net opening approximately equal to the existing cross-sectional area of the present Murray Avenue bridge opening.
- Construction of a high capacity pump station (750-1000 cfs) in the vicinity of the dike to augment the discharge through the gravity outlet of the tidal gates, or to provide a means of draining the watershed when these gates are closed.
- Channel improvement of Turkey Creek between Murray Avenue and the SCLRR crossing. Design specifications of this improvement are given in paragraph 20.c.
- Unblocking the silted SCLRR culvert. In light of the erosion control now being practiced, it is feasible to restore the hydraulic capacity of this culvert.

The extent to which these idealistic improvements can or cannot be incorporated into the final design for flood control largely rests upon the development of an in-depth economic analysis of their costs and a detailed assessment of their social and environmental impacts. These analyses will be undertaken during the second phase of this study. Project design will be finalized contingent upon the outcome of these analyses.

LIST OF TABLES

TABLE 1

HOURLY PRECIPITATION DATA - JUNE 1973

U.S. WEATHER SERVICE HOURLY PRECIPITATION STATION

CHARLESTON AIR FORCE BASE, CHARLESTON, S.C.

Day	A.M. Hour ending at 1	2	3	4	5	6	7	8	9	10	11	12	P.M. Hour ending at 1	2	3	4	5	6	7	8	9	10	11	12
1																								
2																								
3																								
4																								
5																								
6																				T	.08	[illegible]
7												T												
8	.10	[illegible]	T							T	.29		.12	.92	.83	1.33	.13	.01	.03	T	.28	.44	.06	.11
9						T	T		.59	.10	T		T	T	.02	.01	T			T	T			
10						.12	[illegible]		.07	.14	.02	T	.44	.28	.10	.04	T				T	T	.25	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	.14	[illegible]	1.54	.49	.28	.37	.34	.44	.24	.10	.01	T	T			T	.14	[illegible]	.04	.03
12	.02	[illegible]	.21	[illegible]	.03	.06					T			T	T	T	.01	T		T	T			
13	T	T				T	T	T	T	T													T	T
14																		T	.38	1.06	.02	T		
15				[illegible]	.07	.03	.01																	
16																		T						
17																								
18																	1.13	[illegible]	T					
19					.05	T	T				.06	.34				3.89	.58	[illegible]						
20												T	T	T	T									
21											T	T												
22								.32		T	.01	T	T	.07	T	T								
23			T	T	.01	T									.01	.01	.03	.66						
24																			[illegible]					
25															T	T			.10	.04				
26											.01	.07	.32	T		T							[illegible]	
27																								
28			.13	[illegible]	T																			
29																								
30																								

TABLE 2

UNIT HYDROGRAPHS

BASIC DATA

Subbasin Parameter	Turkey Creek Subbasins: A-Hdwtrs to SCLRR Xing	B-between SCLRR & Unnamed Trib	C-Hdwtrs to Mouth of Unnamed Tributary	D-Between Unnamed Trib & Murray Ave	E-Between Murray Ave & Mouth	Goose Creek Subbasin: F-Hdwtrs to Spillway of Dam
DA (mi^2)	2.36	0.20	0.67	0.62	0.15	48.7
L (mi)	2.46	0.83	1.90	1.20	0.42	13.2
L_{ca} (mi)	1.40	0.42	0.91	0.60	0.21	6.8
$(LL_{ca})^{0.3}$	1.45	0.73	1.18	0.91	0.48	3.85
t_r (hrs)	1.00	1.00	1.00	1.00	1.00	1.00
Re(in)	1.00	1.00	1.00	1.00	1.00	1.00
Q_{pr} (cfs)	113.0	15.0	45.0	40.0	10.0	1096
q_{pr} (cfm)	47.9	75.0	67.0	64.5	66.7	22.5
t_{pr} (hrs)	[2.86](2.84)	[1.04](1.56)	(2.08)[1.77]	[1.04](1.60)	[1.04](1.60)	[17.0](17.0)
S_{st}(ft/mi)	11.1	32.1	12.7	30.0	118.8	0.41
C_{tr}	[1.97](1.96)	[1.42](2.14)	[1.76](1.50)	[1.14](1.76)	[2.17](3.33)	[4.41](4.41)
640 C_p	[134] (134)	[115] (115)	[147] (121)	[102] (102)	[102] (102)	[378] (378)

(n) Designates values utilized by computer model HEC-1 to determine subbasin unit hydrograph

[n] Designates values selected by WRE as indicative of unit hydrograph parameters

TABLE 3

UNIT HYDROGRAPH

Time in Hours From Beginning of Rainfall	End of Period Ordinates (CFS) Subbasin A	B	C	D	E	F	Time in Hours From Beginning of Rainfall	End of Period Ordinates (CFS) Subbasin A	B	C	D	E	F	Time in Hours From Beginning of Rainfall	End of Period Ordinates (CFS) Subbasin A	B	C	D	E	F
0	0	0	0	0	0	0	36	6		1	1		[illegible]	72						60
1	23	13	22	21	5	15	37	6		1	1		307	73						57
2	72	15	45	40	10	57	38	5		1	1		276	74						54
3	113	13	41	35	9	113	39	5		0	1		356	75						51
4	111	11	39	32	8	150	40	5			1		327	76						49
5	102	10	34	23	7	[illegible]	41	4			1		300	77						45
5	93	9	30	25	5	357	42	4			0		303	78						43
7	85	8	25	23	6	[illegible]	43	3					237	79						41
8	78	7	23	21	5	[illegible]	44	3					272	80						39
9	71	6	20	13	4	645	45	3					253	81						37
10	65	5	13	16	4	745	46	3					244	82						35
11	59	5	15	15	4	835	47	2					232	83						33
12	55	4	14	13	3	912	48	2					219	84						32
13	50	4	12	12	3	976	49	2					203	85						30
14	45	3	11	10	3	1027	50	2					197	86						28
15	42	3	9	9	2	1064	51	2					187	87						27
16	38	2	8	8	2	1079	52	2					177	88						26
17	35	2	7	7	2	1076	53	1					168	89						24
18	32	2	6	7	2	1054	54	1					159	90						23
19	29	2	6	6	1	1022	55	1					151	91						22
20	27	1	5	5	1	[illegible]	55	1					143	92						21
21	25	1	4	5	1	931	57	1					135	93						19
22	22	1	4	4	1	[illegible]	58	1					123	94						18
23	21	1	3	4	1	[illegible]	59	1					121	95						12
24	19	1	3	3	1	773	60	1					115	96						17
25	17	1	3	3	1	755	61	1					103	97						16
26	16	1	2	3	1	717	62	0					103	98						15
27	14	1	2	2	1	[illegible]	63						99	100						14
28	13	0	2	2	1	[illegible]	64						93	101						13
29	12		2	2	0	610	65						93							
30	11		1	2		578	66						83							
31	10		1	2		544	67						79							
32	9		1	1		519	68						75							
33	8		1	1		492	69						71							
34	8		1	1		[illegible]	70						67							
35	7		1	1		462	71						64							

TABLE 4

RAINFALL-FREQUENCY-DURATION [1]

Return Frequency (years)	Duration in Hours 1	2	3	6	12	24
2	2.2	2.6	2.9	3.2	4.0	4.5
10	2.9	3.6	4.0	4.8	6.0	7.0
25	3.4	4.2	4.7	5.7	6.9	7.9
50	3.7	4.6	5.3	6.4	7.6	9.0
100	4.0	5.2	5.7	7.0	8.6	10.1

[1] Values shown are total rainfall in inches and were extracted from Weather Bureau Technical.Paper No. 40, U.S. Department of Commerce, May 1961.

TABLE 5

TIME DISTRIBUTION OF 24-HOUR DURATION RAINFALL

Data Extracted from Plate 10, Reference 9

Hours from Start of Storm	Percentage of 24-Hour Rainfall in Designated 6-Hour Period
0- 6	7.2%
6-12	16.4%
12-18	66.1%
18-24	10.3%

Data Extracted from Plate 11, Reference 9

Hour of 6-Hour Period	Time Distribution of 1-Hour Rainfall as Percentage of Total 6-Hour Rainfall
1	10%
2	12%
3	15%
4	38%
5	14%
6	11%

Hourly Percentages of Total 24-Hour Rainfall

Hour from Start of Storm	Percent of Total 24-Hour Rainfall Occurring in Designated 1-Hour Period	Hour from Start of Storm	Percent of Total 24-Hour Rainfall Occurring in Designated 1-Hour Period
1	0.72%	13	6.61%
2	0.86%	14	7.93%
3	1.08%	15	9.92%
4	2.74%	16	25.1 %
5	1.01%	17	9.25%
6	0.79%	18	7.30%
7	1.64%	19	1.03%
8	1.97%	20	1.24%
9	2.46%	21	1.54%
10	6.23%	22	3.91%
11	2.30%	23	1.44%
12	1.80%	24	1.13%

TABLE 6

HOURLY DISTRIBUTION OF TOTAL 24-HOUR RAINFALL FOR SELECTED EVENTS

Return Frequency Storm	Total 24-Hour Rainfall (Inches)	Total Rainfall During Hour from Start of Storm (Inches)																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
2	4.52	0.03	0.04	0.05	0.12	0.04	0.04	0.08	0.09	0.11	0.29	0.10	0.08	0.30	0.36	0.45	1.13	0.42	0.33	0.05	0.06	0.07	0.18	0.05	0.05
10	6.95	0.04	0.05	0.07	0.19	0.07	0.06	0.11	0.14	0.17	0.40	0.16	0.13	0.45	0.56	0.69	1.76	0.65	0.51	0.07	0.09	0.11	0.27	0.10	0.08
25	7.88	0.06	0.07	0.08	0.22	0.08	0.06	0.13	0.16	0.19	0.49	0.18	0.14	0.52	0.63	0.78	1.58	0.73	0.57	0.09	0.10	0.12	0.31	0.11	0.09
50	8.95	0.06	0.08	0.10	0.25	0.09	0.07	0.15	0.18	0.22	0.56	0.21	0.16	0.59	0.71	0.89	2.26	0.83	0.65	0.09	0.11	0.14	0.35	0.13	0.10
100	10.07	0.07	0.09	0.11	0.28	0.10	0.08	0.16	0.20	0.25	0.63	0.23	0.18	0.67	0.80	1.00	2.54	0.91	0.73	0.10	0.12	0.16	0.39	0.14	0.11

TABLE 7

STORAGE DATA, GOOSE CREEK IMPOUNDING RESERVOIR, 1917 to 1947, INCLUSIVE
DRAINAGE AREA 42.5 SQUARE MILES — STORAGE CAPACITY 2,781,630,000 GALLONS
FLOW LINE ELEVATION 10.5 FEET

	ELEVATION WATER SURFACE												STORAGE—MILLION GALLONS ABOVE ELEVATION 5.0											
Year	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1917	5.50	6.55	7.91	7.79	8.72	9.29	7.56	10.18	9.52	9.79	9.26	[illegible]	325	565	900	1,169	1,710	1,420	1,915	2,550	2,310	2,320	[illegible]	[illegible]
1918	[illegible]	[illegible]	7.42	[illegible]	6.62	6.15	4.73	4.01	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,199	[illegible]	710	610	410	127	129	[illegible]	[illegible]	[illegible]	[illegible]
1919	[illegible]	[illegible]	[illegible]	10.51	[illegible]	[illegible]	10.53	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,290	[illegible]	[illegible]	[illegible]	2,360	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1920	[illegible]	[illegible]	[illegible]	[illegible]	10.77	10.14	9.50	[illegible]	10.16	10.74	[illegible]	[illegible]	[illegible]	1,760	1,220	[illegible]	2,970	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1921	[illegible]	[illegible]	[illegible]	10.67	10.22	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,910	[illegible]	[illegible]	2,920	2,600	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1922	[illegible]	[illegible]	[illegible]	10.35	[illegible]	[illegible]	10.61	10.55	10.51	[illegible]	10.13	[illegible]	1,750	1,780	[illegible]	[illegible]	2,620	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1923	[illegible]	10.52	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.51	[illegible]	10.21	[illegible]	[illegible]	[illegible]	[illegible]	2,720	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1924	[illegible]	[illegible]	[illegible]	10.52	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1925	[illegible]	[illegible]	10.65	10.50	10.18	[illegible]	9.21	[illegible]	[illegible]	[illegible]	5.66	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	330	63.	[illegible]
1926	[illegible]	[illegible]	[illegible]	[illegible]	10.40	[illegible]	10.60	[illegible]	[illegible]	8.90	[illegible]	[illegible]	2,510	[illegible]	[illegible]	3,020	2,720	2,350	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1927	[illegible]	[illegible]	[illegible]	[illegible]	6.88	[illegible]	5.64	[illegible]	[illegible]	9.56	9.46	[illegible]	1,170	[illegible]	[illegible]	1,020	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1928	[illegible]	8.42	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1929	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1930	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1931	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1932	[illegible]	7.30	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	400	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1933	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,110	[illegible]	[illegible]	[illegible]
1934	[illegible]	[illegible]	[illegible]	[illegible]	10.16	[illegible]	[illegible]	[illegible]	9.01	[illegible]	[illegible]	[illegible]	2,160	2,130	2,120	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1935	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.60	[illegible]	[illegible]	1,650	[illegible]	[illegible]	[illegible]	1,360	1,210	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1936	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1937	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1938	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1939	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1940	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1941	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1942	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1943	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1944	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1945	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1946	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1947	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1948	10.95												3,160											

Minimum Elevation November 24, 1918..........1.08 = Storage 35 Million Gallons

Elevation Center of Intake..............0.0

NOTE:—The reservoir has been supplied with water from the Edisto River-Goose Creek Tunnel since its completion in 1927, and therefore the results shown are not comparable with previous years, nor truly indicative of the rainfall or flow from the Goose Creek drainage area.

Zero of Charleston Public Works (CPW) Datum = -3.0 feet mean sea level

TABLE 8

GOOSE CREEK RESERVOIR

Daily Stages and Rainfall for Selected Storm Events

STORM OF AUGUST 1940

Date	Pool Stage (ft)	Rainfall in Inches
Aug 1, 1940	10.37	0.07
2	10.26	
3	10.26	
4	10.33	
5	10.33	
6	10.31	
7	10.35	1.25
8	10.35	0.03
9	10.34	
10	10.40	0.79
11	13.00	9.70
12	13.20	0.91
13	12.62	1.70
14	12.05	0.10
15	11.70	0.14
16	11.40	0.26
17	11.02	0.02
18	11.02	
19	11.22	2.78
20	11.26	1.02
21	11.20	
22	11.10	
23	11.10	
24	11.15	
25	11.09	
26	11.05	
27	11.00	
28	10.98	
29	10.93	0.14
30	10.95	0.75
31	10.97	0.01
Sep 1	10.86	
2	10.79	
3	10.79	
4	10.73	0.31
5	10.70	0.09
6	10.75	
7	10.62	
8	10.60	
9	10.57	
10	10.58	
11	10.56	
12	10.54	
13	10.52	
14	10.51	
15	10.49	

NOTE: Pool stages are referenced to Charleston Public Works (CPW) datum. Zero of CPW datum equals-3.0 feet mean sea level.

TABLE 9

GOOSE CREEK DAM

AUGUST 1940 STORM

CALIBRATION RESULTS

Date	Elev (Tbl #3) CPW Datum	Storage (Ac-Ft)	HEC-1 Predicted Storage	Percent Difference
1	10.4	8260	8310	+1
2	10.3	8030	8370	4
3	10.3	8030	8410	5
4	10.3	8170	8430	3
5	10.3	8170	8450	3
6	10.3	8130	8460	4
7	10.4	8220	8480	3
8	10.4	8220	8630	5
9	10.3	8190	8710	6
10	10.4	8320	8700	5
11	13.0	16600	9800	-41
12	13.2	17700	15100	-15
13	12.6	14900	13060	-12
14	12.0	12100	11190	-7
15	11.7	11340	10590	-7
16	11.4	10600	10190	-4
17	11.0	9650	9910	3
18	11.0	9650	9720	1
19	11.2	10110	9690	-4
20	11.3	10250	10730	5
21	11.2	10110	10610	5
22	11.1	9880	10220	3
23	11.1	9880	9930	1
24	11.2	10000	9730	-3
25	11.1	10110	9610	-5
26	11.0	9760	9500	-3
27	11.0	9650	9370	-3
28	11.0	9860	9230	-6
29	10.9	9490	9090	-4
30	10.9	9540	8980	-6
			|AVG%D| =	6%

TABLE 10

Streamflow Gaging Stations

Charleston District, Corps of Engineers

Station No.	DA (mi^2)	Mean Elevation (ft msl)	L (mi)	L_{ca} (mi)	S_{st} (ft/mi)	Mean $\overline{X}$	Standard Deviation S	Skew G	Equivalent Years of Record
1483.0	38.1	--	11.0	4.5	13.4	2.410	0.305	1.580	27.4
1089.6	15.0	55	4.0	2.5	4.0	2.655	0.283	-0.571	23.7
1096.4	16.0	52	4.7	3.0	4.1	2.665	0.327	0.106	21.7
1100.2	3.0	90	3.9	1.6	10.4	2.235	0.385	-0.107	25.5
1270.0	110.0	428	19.9	6.7	10.3	3.378	0.357	0.508	41.0
1273.9	0.9	--	--	--	--	2.188	0.484	-1.702	34.6
1282.6	15.4	514	7.6	3.7	27.1	3.048	0.244	0.820	31.7
1294.4	17.0	435	5.5	2.6	21.1	2.942	0.188	0.642	25.9
1305.0	64.0	90+	15.3	8.4	7.7	2.682	0.304	2.478	38.0
1309.0	108.0	462	26.3	12.6	8.2	2.937	0.125	-0.805	36.1
1309.1	173.0	433	36.8	14.7	7.6	2.960	0.227	1.757	39.6
1311.5	28.0	101	12.0	6.1	4.9	2.587	0.391	-1.002	38.6
1322.3	6.2	183	5.6	2.7	9.7	2.012	0.283	0.514	26.8
1335.9	4.7	350	8.5	2.4	26.5	1.859	0.246	-0.516	26.1
1339.6	40.0	278	1.3	8.0	7.8	2.587	0.284	-1.435	40.8
1343.8	16.0	171	6.4	4.3	4.5	2.335	0.197	-0.409	25.0
1695.5	136.0	318	17.4	9.4	10.3	2.954	0.135	-0.291	27.3
1696.3	10.0	304	3.7	1.5	30.6	2.226	0.395	1.248	31.0
1725.0	198.0	462	33.8	16.9	7.4	3.175	0.227	-0.229	34.8
1765.0	203.0	175	21.5	7.8	5.0	3.233	0.241	0.131	29.4
1716.8	17.4	40	5.2	2.1	5.8	2.400	0.386	-0.210	32.3

TABLE 11

LAND USE/IMPERVIOUS AREA CORRELATIONS

Land Use	No. of Areas Evaluated	Total Area Evaluated (1000 ft^2)	Size (1000 ft^2)		% Impervious Streets Only			% Impervious Streets, Drives, Walks, Parking			% Impervious Streets, Drives, Walks, Roofs, Parking		
			Min.	Max.	Min.	Max.	Ave.	Min.	Max.	Ave.	Min.	Max.	Ave.
Single Family	73	23,490	100.7	1116	4.4	20.2	12.6	8.0	28.6	13.8	13.2	52.9	24.3
Medium Density Residential	15	3,305	60.2	397	7.5	19.6	11.3	12.2	61.9	23.2	19.2	65.0	34.2
High Density	46	20,356	112.0	2000	5.4	36.3	11.8	11.9	62.9	31.6	30.9	71.0	47.8
Schools	9	8,339	165.5	1900	3.8	13.5	5.9	14.9	36.5	20.5	23.2	49.4	30.8
Industrial	5	1,417	143.0	483	42.0	85.1	62.6	76.0	96.6	87.3	86.9	97.6	92.6
Commercial	15	4,339	86.3	1800	15.3	97.2	44.0	61.2	99.6	78.2	64.7	99.7	82.8
Vacant/Open	--	--	--	--	--	--	8.0	--	--	8.0	--	--	8.0
Wetlands	--	--	--	--	--	--	99.0	00	00	99.0	--	--	99.0
TOTAL	163	61,246											

TABLE 12

PERCENT DAMAGE TO TYPICAL STRUCTURE INCLUDING FURNISHINGS

DEPTH OF WATER		ABOVE GROUND ELEV. (Feet)									
		1	2	3	4	5	6	7	8	9	10
		ABOVE FLOOR ELEV. (Feet)									
		-1	0	1	2	3	4	5	6	7	8
ELEVATION IN TENTHS	.0	0.2	1.5	20.9	40.2	52.0	60.6	67.8	73.9	77.9	80.0
	.1	0.2	2.1	23.0	41.8	52.8	61.4	68.7	74.2	78.2	80.0
	.2	0.3	3.2	25.1	43.0	53.9	62.0	69.1	74.8	78.2	80.0
	.3	0.4	5.3	27.5	44.2	54.8	62.9	70.0	75.2	78.7	80.0
	.4	0.5	7.9	29.0	45.6	55.6	63.8	70.4	75.7	79.0	80.0
	.5	0.6	10.2	30.7	46.7	56.5	64.3	71.1	76.1	79.2	80.0
	.6	0.7	12.0	33.0	47.9	57.1	65.1	71.7	76.3	79.3	80.0
	.7	0.9	14.0	35.0	49.0	58.0	66.0	72.2	76.9	79.6	80.0
	.8	1.0	15.9	37.0	50.0	59.0	66.6	72.9	77.1	79.8	80.0
	.9	1.2	18.3	38.6	51.0	59.8	67.2	73.3	77.5	79.9	80.0

TABLE 13

VALUE OF COMMERCIAL/BUSINESS AND
PUBLIC PROPERTIES LOCATED IN TURKEY CREEK FLOOD PLAIN

Building No. =	615-A
Type of Business =	Hanahan Public Service
Building Const. Of =	Brick
Building Cost =	$2,000
Inventory Cost =	$12,000
Finish Floor Elev. =	8.88
Maximum Elev.=	9.8

Building No. =	201-A
Type of Business =	Hanahan Public Service
Building Const. Of =	Brick
Building Cost =	$2,000
Inventory Cost =	$12,000
Finish Floor Elev. =	5.81
Maximum Elev.	10.0

Building No. =	714
Type of Business =	Office & Swimming Pool, Hawthorne Trailer Park
Building Const. Of =	Block
Building Cost =	$30,000
Inventory Cost =	$5,500
Finish Floor Elev.	15.46
Maximum Elev.	15.7

Building No. =	702
Type of Business =	Restaurant
Building Const. Of =	Brick & Block
Building Cost =	$71,000
Inventory Cost =	$48,000
Finish Floor Elev. =	14.97
Maximum Elev. =	15.7

Building No. =	703
Type of Business =	Carpet Wholesalers
Building Const. Of =	Brick & Block
Building Cost =	$365,000
Inventory Cost =	$300,000
Finish Floor Elev. =	14.25
Maximum Elev. =	15.7

Building No. =	704
Type of Business =	Finance Co.
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$1,500
Finish Floor Elev. =	13.41
Maximum Elev.=	15.7

Building No. =	705
Type of Business =	Tax Service
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$2,000
Finish Floor Elev. =	12.50
Maximum Elev.	15.7

Building No. =	706
Type of Business =	Beauty Salon
Building Const. Of =	Wood
Building Cost=	$38,500
Inventory Cost =	$8,100
Finish Floor Elev. =	12.23
Maximum Elev.=	15.7

Building No. =	707
Type of Business =	Real Estate Co.
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$3,000
Finish Floor Elev.	11.97
Maximum Elev.	15.7

Building No. =	708
Type of Business =	Printing & Office Supplies
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$35,000
Finish Floor Elev. =	12.43
Maximum Elev. =	15.7

Building No. =	709
Type of Business =	Law Office
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$5,000
Finish Floor Elev.	13.08
Maximum Elev. =	15.7

Building No. =	710
Type of Business =	Vacant
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	None
Finish Floor Elev. =	12.85
Maximum Elev. =	15.7

Building No. =	711
Type of Business =	Vacant
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	None
Finish Floor Elev. =	13.15
Maximum Elev. =	15.7

Building No. =	712
Type of Business =	Magnetic Sign Co.
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$15,000
Finish Floor Elev. =	14.56
Maximum Elev. =	15.7

Building No. =	713
Type of Business =	Charles Towne Carpet Co.
Building Const. Of =	Wood
Building Cost =	$38,500
Inventory Cost =	$2,000
Finish Floor Elev. =	15.47
Maximum Elev. =	15.7

Building No. =	1007
Type of Business =	Motel
Building Const. Of =	Block & Brick
Building Cost =	900,000
Inventory Cost =	100,000
Finish Floor Elev. =	23.67
Maximum Elev. =	25.9

TABLE 14

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage: Residential   Damage Stage: 5.3 Ft MSL (Zero Damage)

Reach Number: Reach 1   Fluvial Flood

Condition: Turkey Creek - Charleston, South Carolina

Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			370.0	1,850
200	.0050		9.7	370		
		.0050			305.0	1,525
100	.0100		9.2	240		
		.0025			215.0	538
80	.0125		9.0	190		
		.0042			155.0	651
60	.0167		8.6	120		
		.0083			85.0	706
40	.0250		8.2	50		
		.0083			40.0	332
30	.0333		8.0	30		
		.0167			18.5	309
20	.0500		7.6	7		
		.0167			5.0	84
15	.0667		7.3	3		
		.0333			2.0	67
10	.1000		7.0	1		
		.1000			0.75	75
5	.2000		6.3	0.5		
		.1333			0.35	47
3	.3333		5.7	0.2		
		.1667			0.15	25
2	.5000		5.3	0.1		
		.5000			0.05	25
1	1.0000		4.7	0		
		1.0000			0.0	0
.5	2.0000					
					TOTAL	$ 6,234

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 15

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Public Damage Stage 3.8 Ft MSL (Zero Damage)

Reach Number Reach 1 Fluvial Flood

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			10.40	52
200	.0050		9.7	10.4		
		.0050			9.65	48
100	.0100		9.2	8.9		
		.0025			8.60	22
80	.0125		9.0	8.3		
		.0042			7.75	33
60	.0167		8.6	7.2		
		.0083			6.70	56
40	.0250		8.2	6.2		
		.0083			5.90	49
30	.0333		8.0	5.6		
		.0167			5.05	84
20	.0500		7.6	4.5		
		.0167			4.10	68
15	.0667		7.3	3.7		
		.0333			3.25	108
10	.1000		7.0	2.8		
		.1000			2.15	215
5	.2000		6.3	1.5		
		.1333			1.05	140
3	.3333		5.7	0.6		
		.1667			0.40	67
2	.5000		5.3	0.2		
		.5000			0.10	50
1	1.0000		4.7	0		0
		1.0000				
.5	2.0000					
					TOTAL	$ 992

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 16

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Residential

Damage Stage 5.3 Ft MSL (Zero Damage)

Reach Number Reach 1

Tidal Surge

Condition Turkey Creek - Charlston, South Carolina

Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			2110.0	10,550
200	.0050		13.8	2110		
		.0050			1570.0	7,850
100	.0100		11.7	1030		
		.0025			920.0	2,300
80	.0125		11.1	810		
		.0042			700.0	2,940
60	.0167		10.4	590		
		.0083			450.0	3,735
40	.0250		9.5	310		
		.0083			240.0	1,992
30	.0333		8.9	170		
		.0167			100.0	1,670
20	.0500		8.0	30		
		.0167			17.0	284
15	.0667		7.4	4		
		.0333			2.35	78
10	.1000		6.6	0.7		
		.1000			0.45	45
5	.2000		5.6	0.2		
		.1333			0.10	13
3	.3333		5.1	0.0		
		.1667			0.0	0
2	.5000		4.8	0.0		
		.5000			0.0	0
1	1.0000		4.6	0.0		
		1.0000			0.0	0
.5	2.0000					
					TOTAL	$ 31,457

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 17

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Public   Damage Stage 3.8 Ft MSL (Zero Damage)

Reach Number Reach 1   Tidal Surge

Condition Turkey Creek - Charleston, South Carolina

Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	- Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			17.8	89
200	.0050		13.8	17.8		
		.0050			16.2	81
100	.0100		11.7	14.6		
		.0025			14.05	35
80	.0125		11.1	13.5		
		.0042			12.85	54
60	.0167		10.4	12.2		
		.0083			11.0	91
40	.0250		9.5	9.8		
		.0083			8.9	74
30	.0333		8.9	8.0		
		.0167			6.8	114
20	.0500		8.0	5.6		
		.0167			4.8	80
15	.0667		7.4	4.0		
		.0333			3.0	100
10	.1000		6.6	2.0		
		.1000			1.25	125
5	.2000		5.6	0.5		
		.1333			0.30	40
3	.3333		5.1	0.1		
		.1667			0.10	17
2	.5000		4.8	0.1		
		.5000			0.05	25
1	1.0000		4.6	0.0		
		1.0000				
.5	2.0000					
					TOTAL	$ 925

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 18

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Residential Damage Stage 11.0 Ft MSL (Zero Damage)

Reach Number Reach 2 Fluvial Stage

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			165.0	825
200	.0050		14.4	165		
		.0050			150.0	750
100	.0100		13.9	135		
		.0025			116.5	291
80	.0125		13.3	98		
		.0042			74.0	311
60	.0167		12.6	50		
		.0083			26.5	220
40	.0250		11.4	3		
		.0083			1.5	12
30	.0333		11.0	0		
		.0167				0
20	.0500		10.2	0		
		.0167				0
15	.0667		9.7	0		
		.0333				0
10	.1000		9.1	0		
		.1000				0
5	.2000		8.2	0		
		.1333				0
3	.3333		7.8	0		
		.1667				0
2	.5000		7.1	0		
		.5000				0
1	1.0000					
		1.0000				0
.5	2.0000					
					TOTAL	$ 2,409

SAN 120, 4/26/65
*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 19

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Commercial Damage Stage 11.0 Ft MSL (Zero Damage)

Reach Number Reach 2 Fluvial Stage

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			203.0	1,015
200	.0050		14.4	203		
		.0050			179.0	895
100	.0100		13.9	155		
		.0025			122.5	306
80	.0125		13.3	90		
		.0042			59.5	250
60	.0167		12.6	29		
		.0083			14.9	124
40	.0250		11.4	0.8		
		.0083			0.4	3
30	.0333		11.0	0		
		.0167				0
20	.0500		10.2	0		
		.0167				0
15	.0667		9.7	0		
		.0333				0
10	.1000		9.1	0		
		.1000				0
5	.2000		8.2	0		
		.1333				0
3	.3333		7.8	0		
		.1667				0
2	.5000		7.1	0		
		.5000				0
1	1.0000					
		1.0000				0
.5	2.0000					
					TOTAL	$ 2,593

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 20

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Residential Damage Stage 11.0 Ft MSL (Zero Damage)

Reach Number Reach 2 Tidal Surge

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			130.00	650
200	.0050		13.8	130		
		.0050			68.75	344
100	.0100		11.7	7.5		
		.0025			3.75	9
80	.0125		11.1	0		
		.0042				
60	.0167		10.4	0		
		.0083				
40	.0250		9.5	0		
		.0083				
30	.0333		8.9	0		
		.0167				
20	.0500		8.0	0		
		.0167				
15	.0667		7.4	0		
		.0333				
10	.1000		6.6	0		
		.1000				
5	.2000		5.6	0		
		.1333				
3	.3333		5.1	0		
		.1667				
2	.5000		4.8	0		
		.5000				
1	1.0000		4.6	0		
		1.0000				
.5	2.0000					
					TOTAL	$ 1,003

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 21

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Commercial Damage Stage 11.0 Ft MSL (Zero Damage)

Tidal Surge

Reach Number Reach 2

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			145.00	725
200	.0050		13.8	145		
		.0050			73.0	365
100	.0100		11.7	1		
		.0025			0.5	1
80	.0125		11.1	0		
		.0042				
60	.0167		10.4	0		
		.0083				
40	.0250		9.5	0		
		.0083				
30	.0333		8.9	0		
		.0167				
20	.0500		8.0	0		
		.0167				
15	.0667		7.4	0		
		.0333				
10	.1000		6.6	0		
		.1000				
5	.2000		5.6	0		
		.1333				
3	.3333		5.1	0		
		.1667				
2	.5000		4.8	0		
		.5000				
1	1.0000		4.6	0		
		1.0000				
.5	2.0000					
					TOTAL	$ 1,091

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 22

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage: Commercial — Damage Stage: 18.9 Ft MSL (Zero Damage)

Reach Number: Reach 3 — Fluvial Stage

Condition: Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	- Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			33.0	165
200	.0050		20.8	33		
		.0050			28.0	140
100	.0100		20.2	23		
		.0025			16.5	41
80	.0125		19.4	10		
		.0042			5.0	21
60	.0167		18.4	0		
		.0083				
40	.0250		17.0	0		
		.0083				
30	.0333		16.4	0		
		.0167				
20	.0500		15.5	0		
		.0167				
15	.0667		15.0	0		
		.0333				
10	.1000		14.4	0		
		.1000				
5	.2000		13.2	0		
		.1333				
3	.3333		12.7	0		
		.1667				
2	.5000		12.3	0		
		.5000				
1	1.0000					
		1.0000				
.5	2.0000					
					TOTAL	$ 367

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 23

AVERAGE ANNUAL DAMAGE COMPUTATION

Type of Damage Commercial   Damage Stage 18.9 Ft MSL (Zero Damage)

Reach Number Reach 3   Tidal Surge

Condition Turkey Creek - Charleston, South Carolina
Existing Conditions

Frequency in years	Probable Occurrence	Incremental Probability	Elevation of WS (msl)	Damages in $1,000	- Average	Damage Increment $
		.0010				
1,000	.0010					
		.0010				
500	.0020					
		.0013				
300	.0033					
		.0050*			0	0
200	.0050		13.8	0		
		.0050			0	
100	.0100		11.7	0		
		.0025			0	
80	.0125		11.1	0		
		.0042			0	
60	.0167		10.4	0		
		.0083			0	
40	.0250		9.5	0		
		.0083			0	
30	.0333		8.9	0		
		.0167			0	
20	.0500		8.0	0		
		.0167			0	
15	.0667		7.4	0		
		.0333			0	
10	.1000		6.6	0		
		.1000			0	
5	.2000		5.6	0		
		.1333			0	
3	.3333		5.1	0		
		.1667			0	
2	.5000		4.8	0		
		.5000			0	
1	1.0000		4.6	0		
		1.0000			0	
.5	2.0000					
					TOTAL	$ 0

SAN 120, 4/26/65

*Damage computations are based on events with a return period frequency less than or equal to 200 years.

TABLE 24

SUMMARY OF FLOOD DAMAGES TURKEY CREEK

DAMAGE TYPE	REACH 1				REACH 2				REACH 3				BASIN TOTAL
	PUBLIC	RESIDENTIAL	COMMERCIAL	TOTAL	PUBLIC	RESIDENTIAL	COMMERCIAL	TOTAL	PUBLIC	RESIDENTIAL	COMMERCIAL	TOTAL	
AVERAGE ANNUAL FLUVIAL FLOOD	$ 990	$ 6230	$ 0	$ 7230	$ 0	$ 2409	$ 2593	$ 5002	$ 0	$ 0	$ 367	$ 367	$ 12,599
AVERAGE ANNUAL TIDAL SURGE	920	31460	0	32380	0	1000	1090	2090	0	0	0	0	34,470
TOTAL AVERAGE ANNUAL	1910	37690	0	39610	0	3409	3683	7092	0	0	367	367	47,069
STANDARD PROJECT FLOOD	11200	619500	0	630700	0	230100	334000	546100	0	0	48200	48200	1,243,000
STANDARD PROJECT HURRICANE	21900	4835500	0	4857400	0	337900	1002400	1340300	0	0	75000	75000	6,272,700

TABLE 25

AVERAGE WATER SURFACE ELEVATIONS
FOR REACHES DURING 100-YEAR FLUVIAL STORM

Reach	Existing Conditions	Alternatives 1	2	3	4
1	9.2	9.2	9.0	10.1	9.1
2	13.9	11.6	11.1	13.5	11.7
3	20.2	20.2	19.6	20.2	20.2

Alternatives:

1. SCL Railroad culvert fully open (under natural conditions one culvert partially silted.
2. Channel improvement from Murray Avenue to SCL Railroad with railroad culvert fully open.
3. Levees from mouth of Turkey Creek to Highway 52, with railroad culvert fully open.
4. Combination of 1, 2 and 3.

LIST OF FIGURES

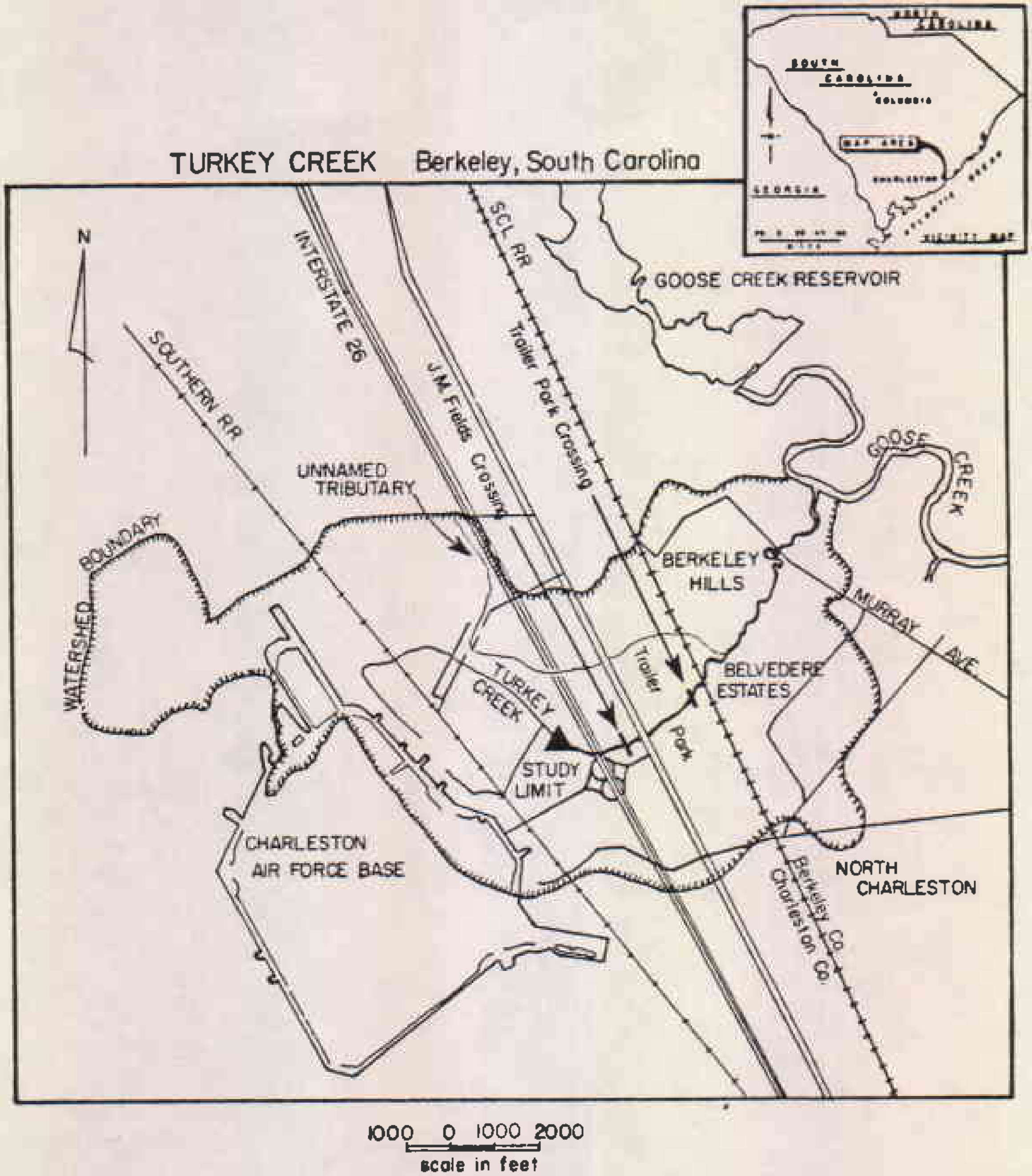


Figure 1

STUDY AREA MAP AND PROMINENT FEATURES

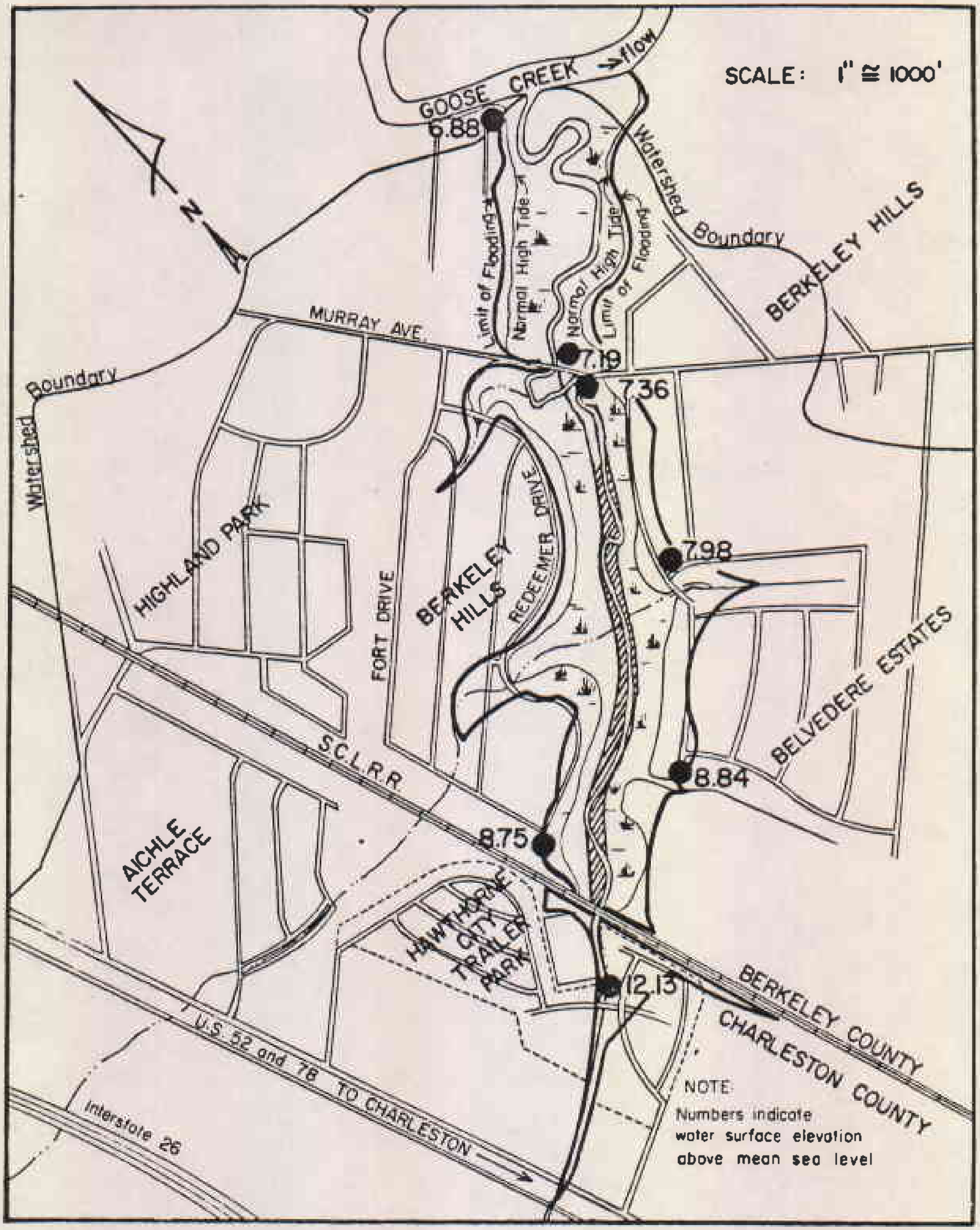
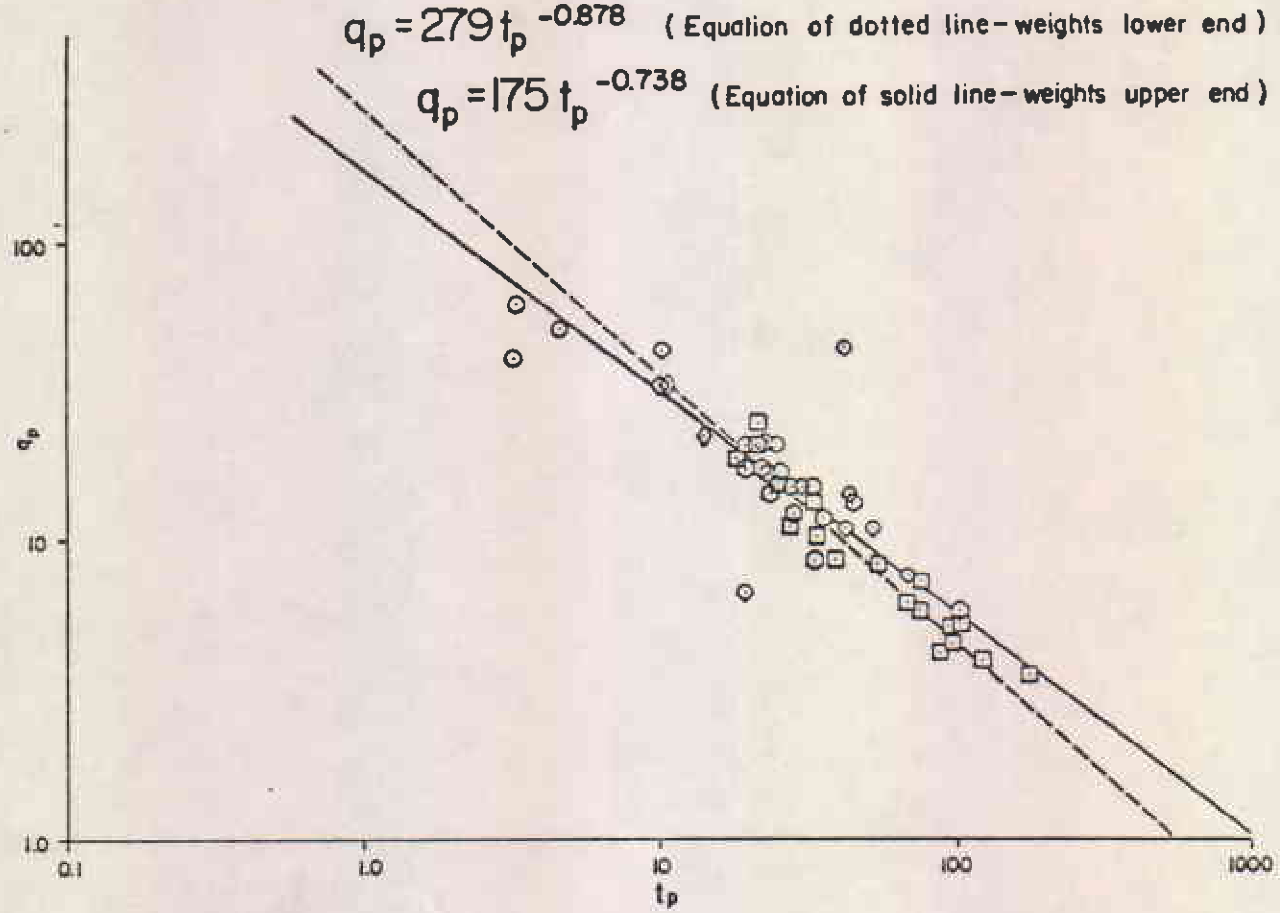


Figure 2

TURKEY CREEK FLOOD OF JUNE 11,1973 , PLAN VIEW



LEGEND

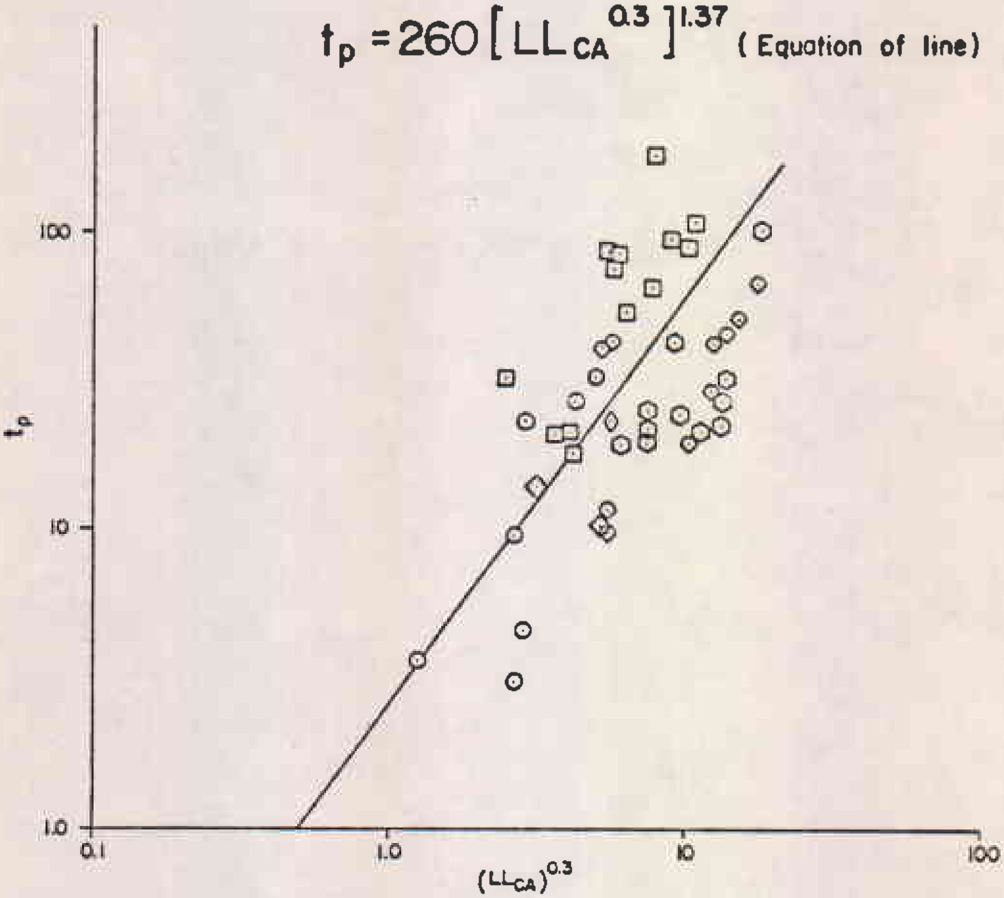
- ⊙ Data points extracted from regional frequency analysis data provided by Charleston District Corps of Engineers
- ⊡ Data points extracted from Wilmington District Corps of Engineers Report "Regional Flood Frequency Study of North Carolina Coastal Plain"
- ◇ Data points extracted from unit hydrograph data provided by Mobile District Corps of Engineers for coastal basins
- ⬡ Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Division, Volume II
- ⊙ Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Divison, Volume IV

Values shown are for 6-hour unit hydrographs

Figure 3

q_p VERSUS t_p

SELECTED UNIT HYDROGRAPHS, SOUTH ATLANTIC DIVISION, CORPS OF ENGINEERS



LEGEND

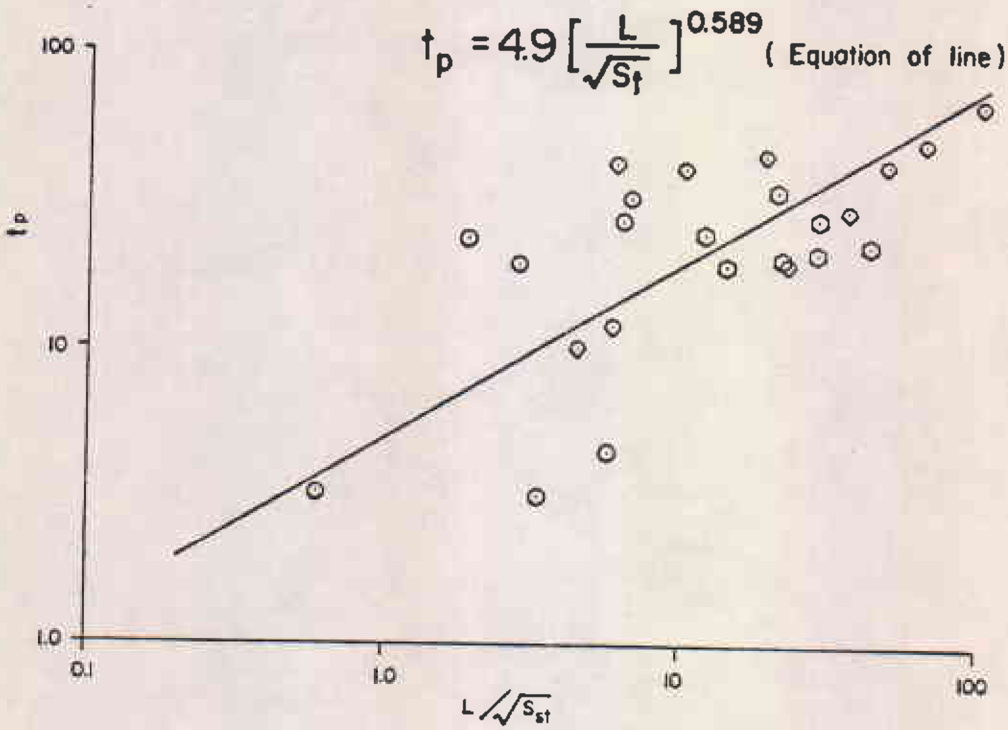
- ⊙ Data points extracted from regional frequency analysis data provided by Charleston District Corps of Engineers
- ⊡ Data points extracted from Wilmington District Corps of Engineers Report "Regional Flood Frequency Study of North Carolina Coastal Plain"
- ◇ Data points extracted from unit hydrograph data provided by Mobile District Corps of Engineers for coastal basins
- Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Division, Volume II
- Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Divison, Volume IV

Values shown are for 6-hour unit hydrographs

Figure 4

t_p VERSUS $(LL_{CA})^{0.3}$

SELECTED WATERSHEDS, SOUTH ATLANTIC DIVISION, CORPS OF ENGINEERS



LEGEND

Data points extracted from regional frequency analysis data provided by Charleston District Corps of Engineers

Data points extracted from Wilmington District Corps of Engineers Report "Regional Flood Frequency Study of North Carolina Coastal Plain"

Data points extracted from unit hydrograph data provided by Mobile District Corps of Engineers for coastal basins

Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Division, Volume II

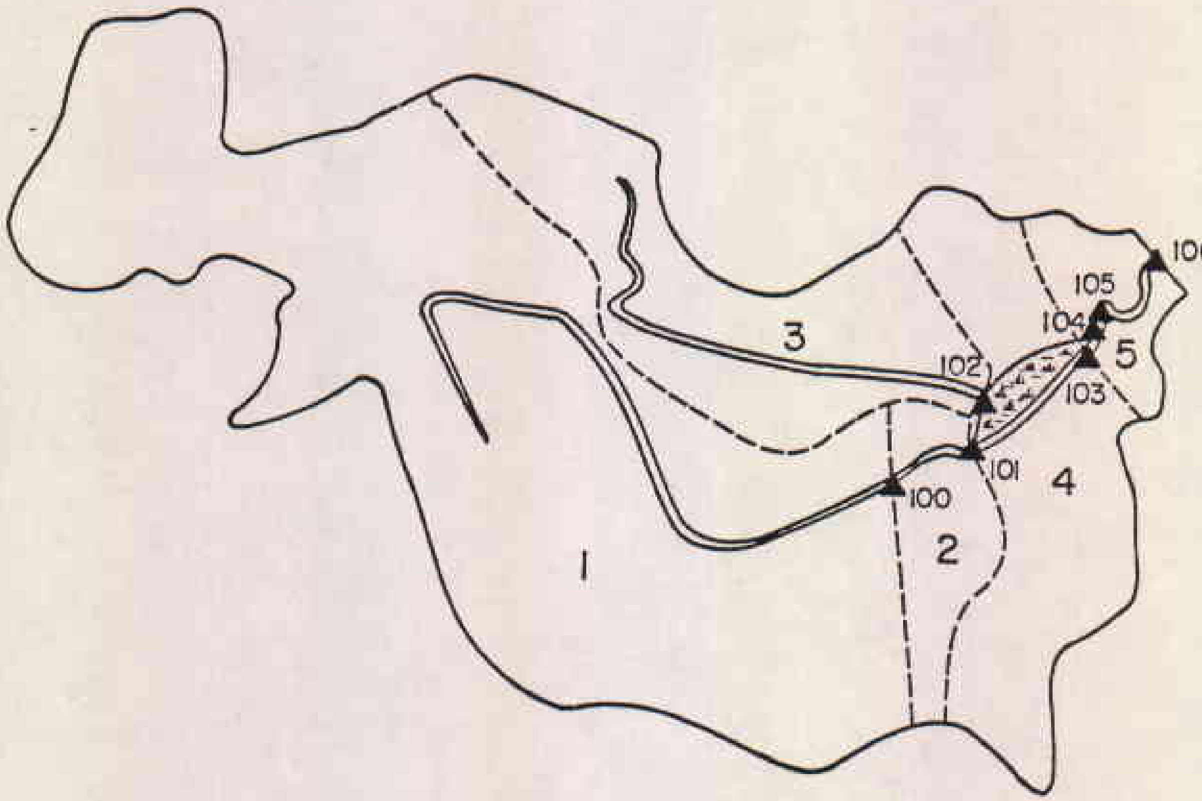
Data points extracted from Civil Works Investigations' Unit Hydrograph Compilations for the South Atlantic Divison, Volume IV

Values shown are for 6-hour unit hydrographs

Figure 5

t_p VERSUS $L\sqrt{S_{ST}}$

SELECTED WATERSHEDS, SOUTH ATLANTIC DIVISION, CORPS OF ENGINEERS



SUB-BASIN NO.	DESCRIPTION
1	HDWTRS TO SCLRR CROSSING
2	SCLRR CROSSING TO CONFLUENCE UNNAMED TRIBUTARY
3	DA UNNAMED TRIBUTARY
4	CONFLUENCE UNNAMED TRIBUTARY TO MURRAY AVENUE
5	MURRAY AVENUE TO MOUTH

SELECTED NODE NO.	DESCRIPTION
100	TURKEY CREEK AT SCLRR CULVERTS
101	TURKEY CREEK AT UNNAMED TRIBUTARY
102	CONFLUENCE TURKEY CREEK AND TRIBUTARY
103	TURKEY CREEK AT MURRAY AVENUE
104	COMPUTATIONAL NODE
105	COMPUTATIONAL NODE
106	CONFLUENCE TURKEY CREEK AND GOOSE CREEK

Figure 6

TURKEY CREEK SCLRR ROUTE MODEL

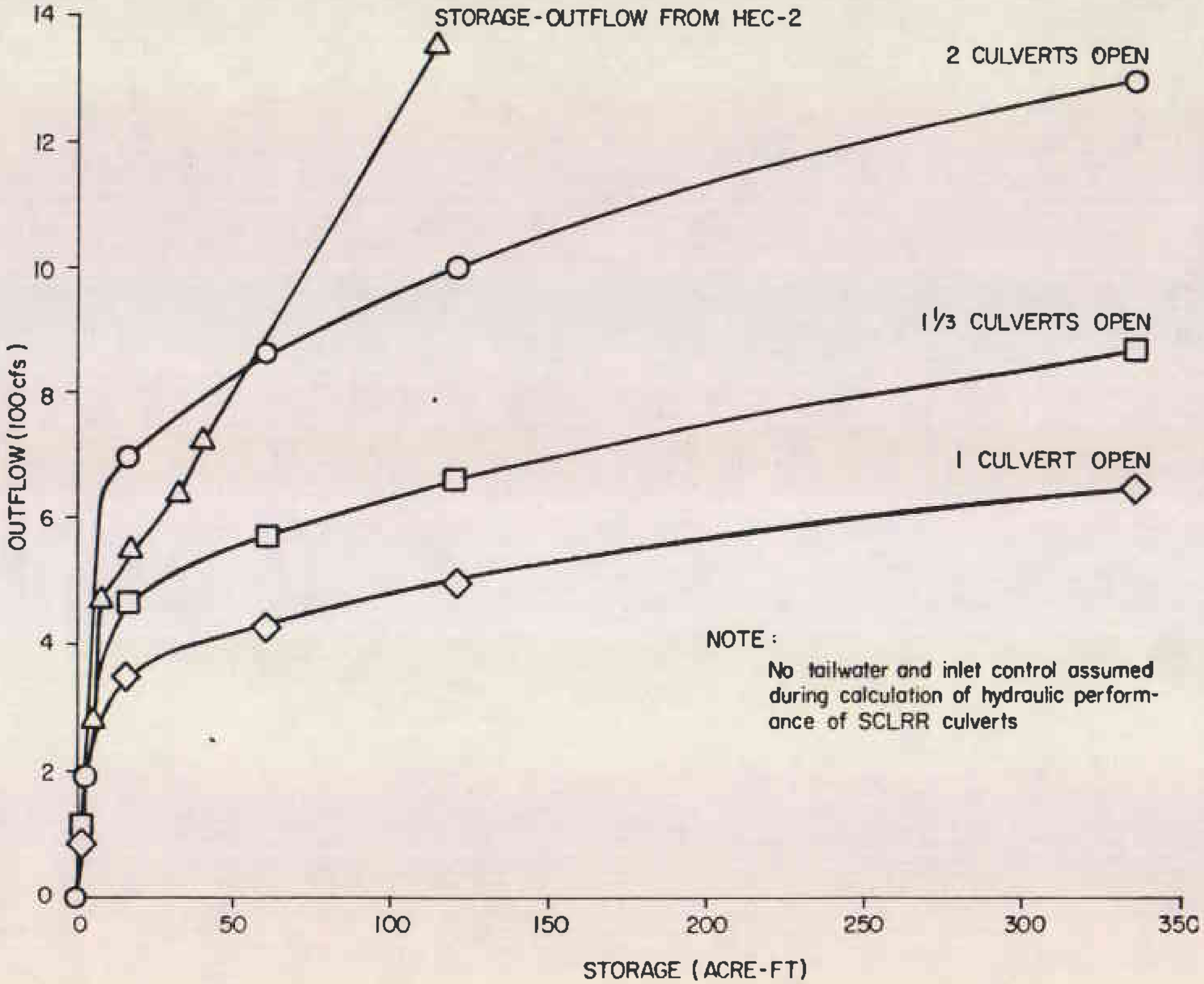


Figure 7
TURKEY CREEK SCLRR CULVERT DISCHARGE RATING CURVE

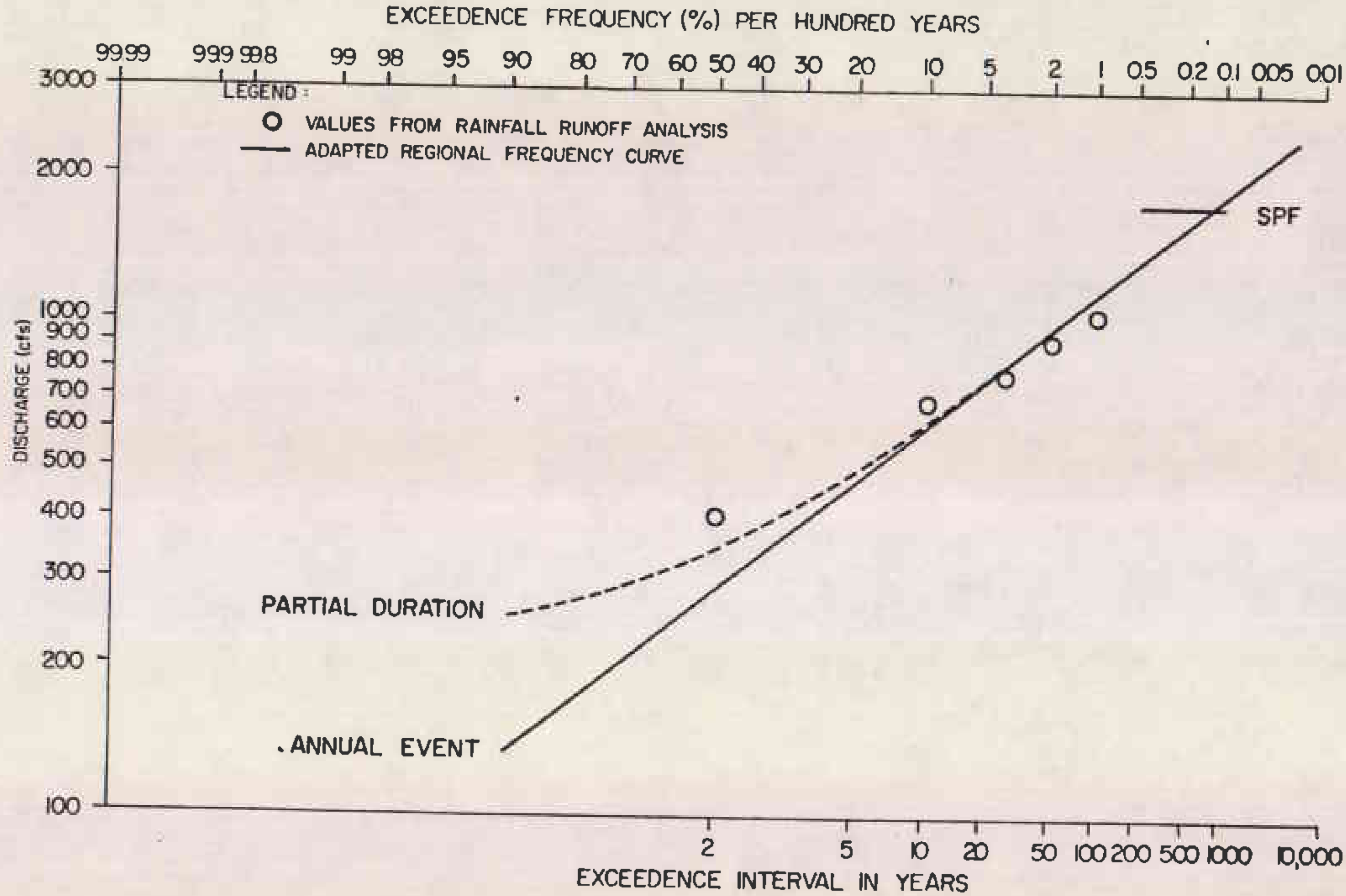


Figure 8

TURKEY CREEK AT MOUTH PEAK DISCHARGE FREQUENCY

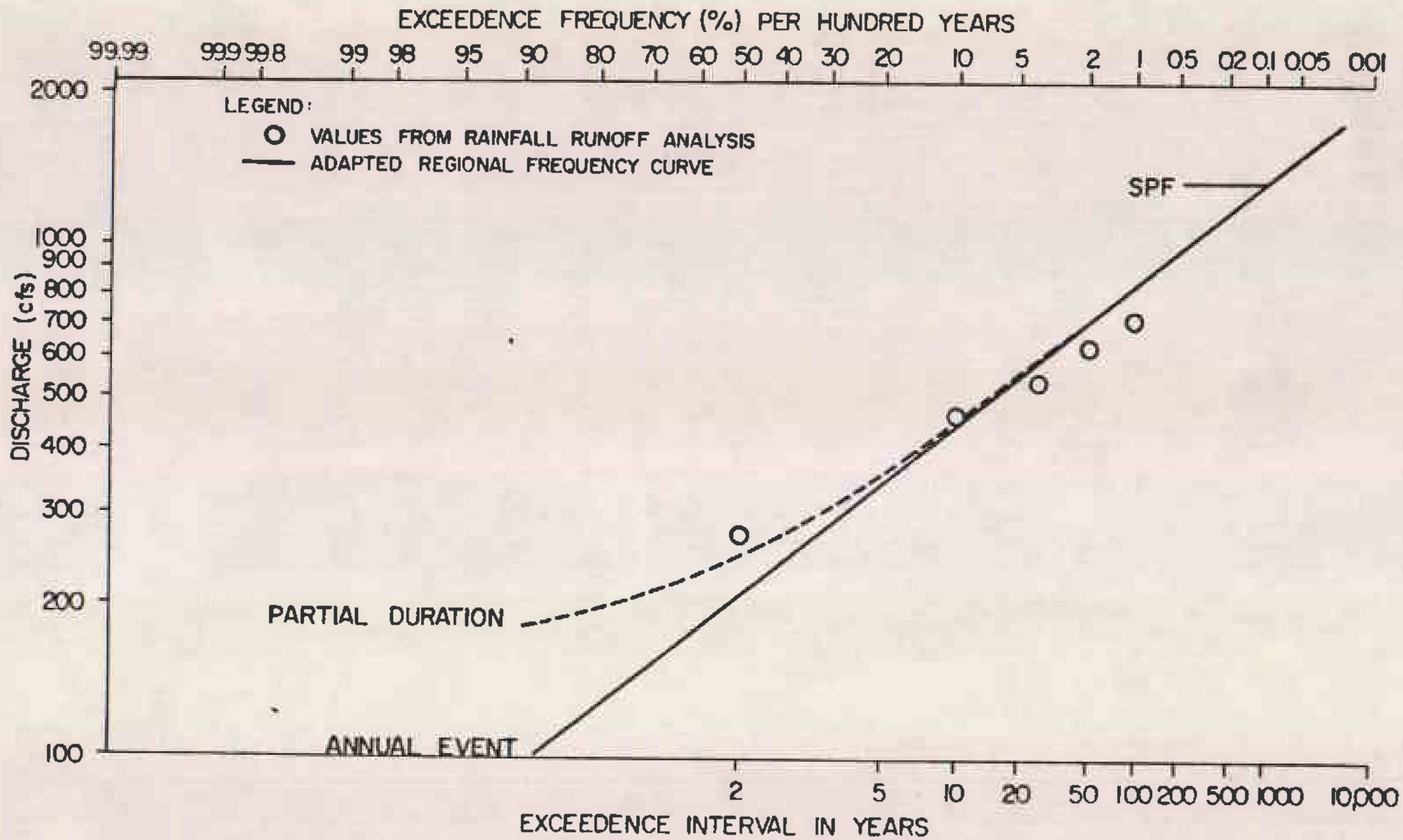


Figure 9

TURKEY CREEK AT SCLRR CROSSING PEAK DISCHARGE FREQUENCY

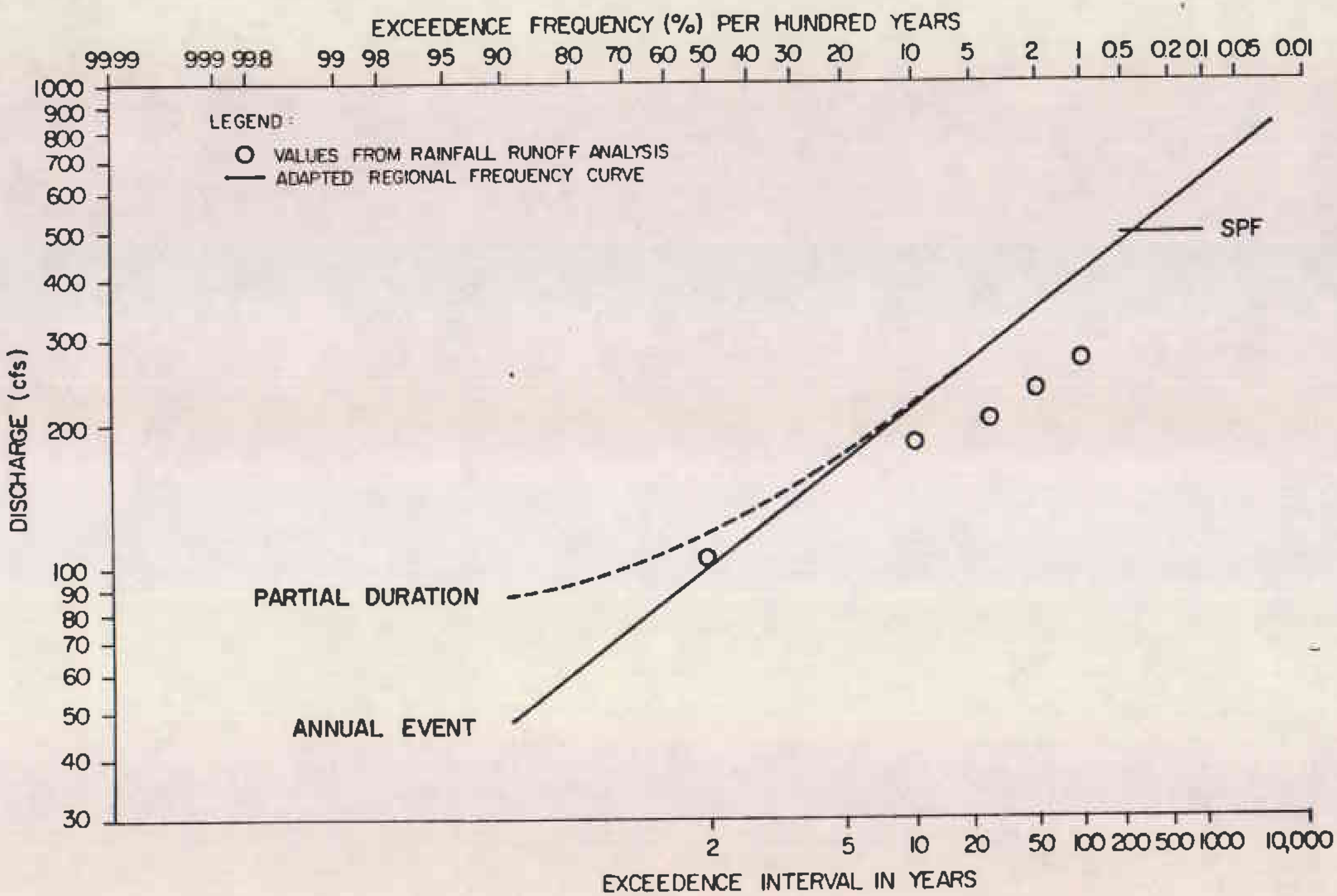


Figure 10

UNNAMED TRIBUTARY AT MOUTH PEAK DISCHARGE FREQUENCY

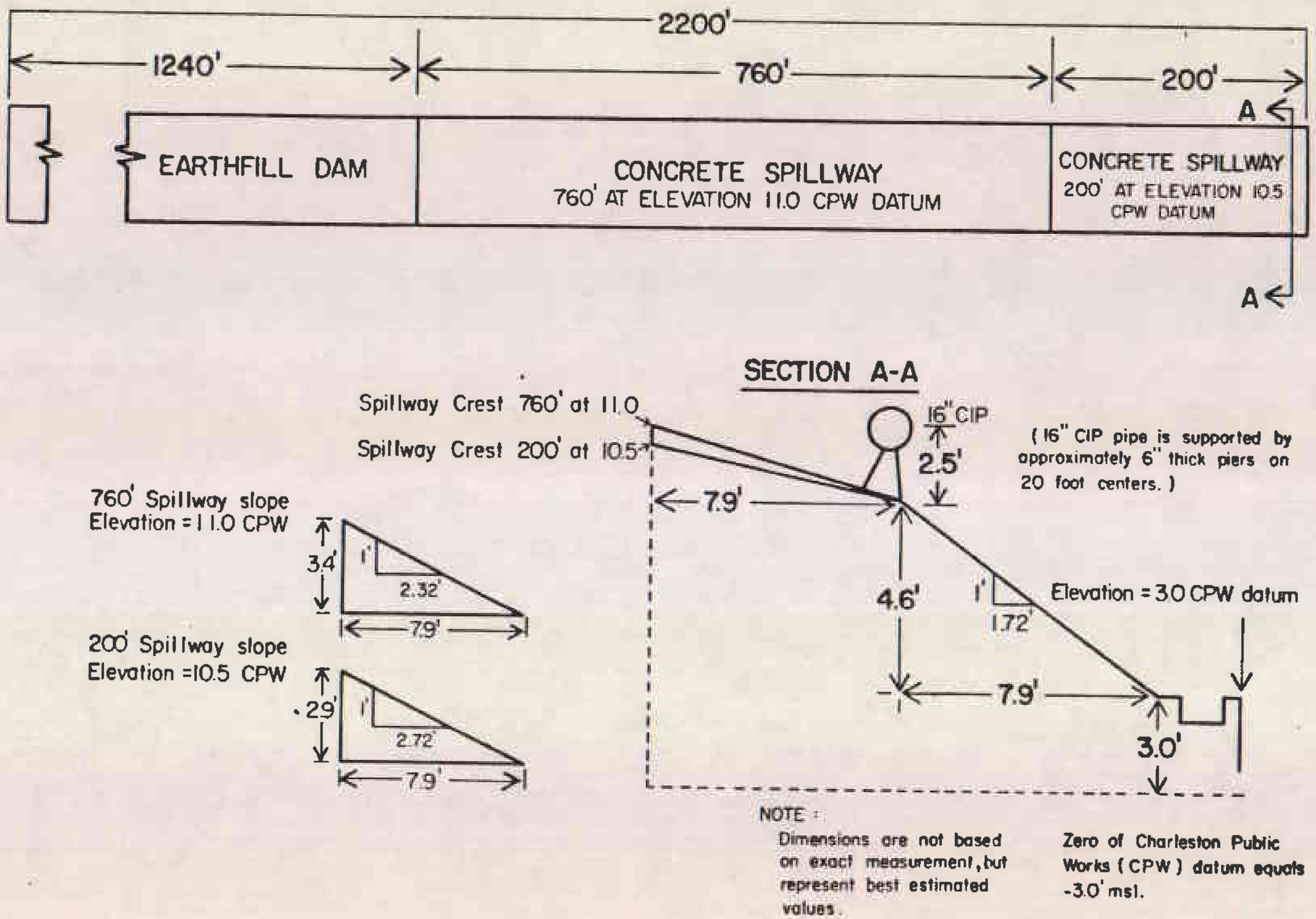


Figure 11
GOOSE CREEK RESERVOIR DAM

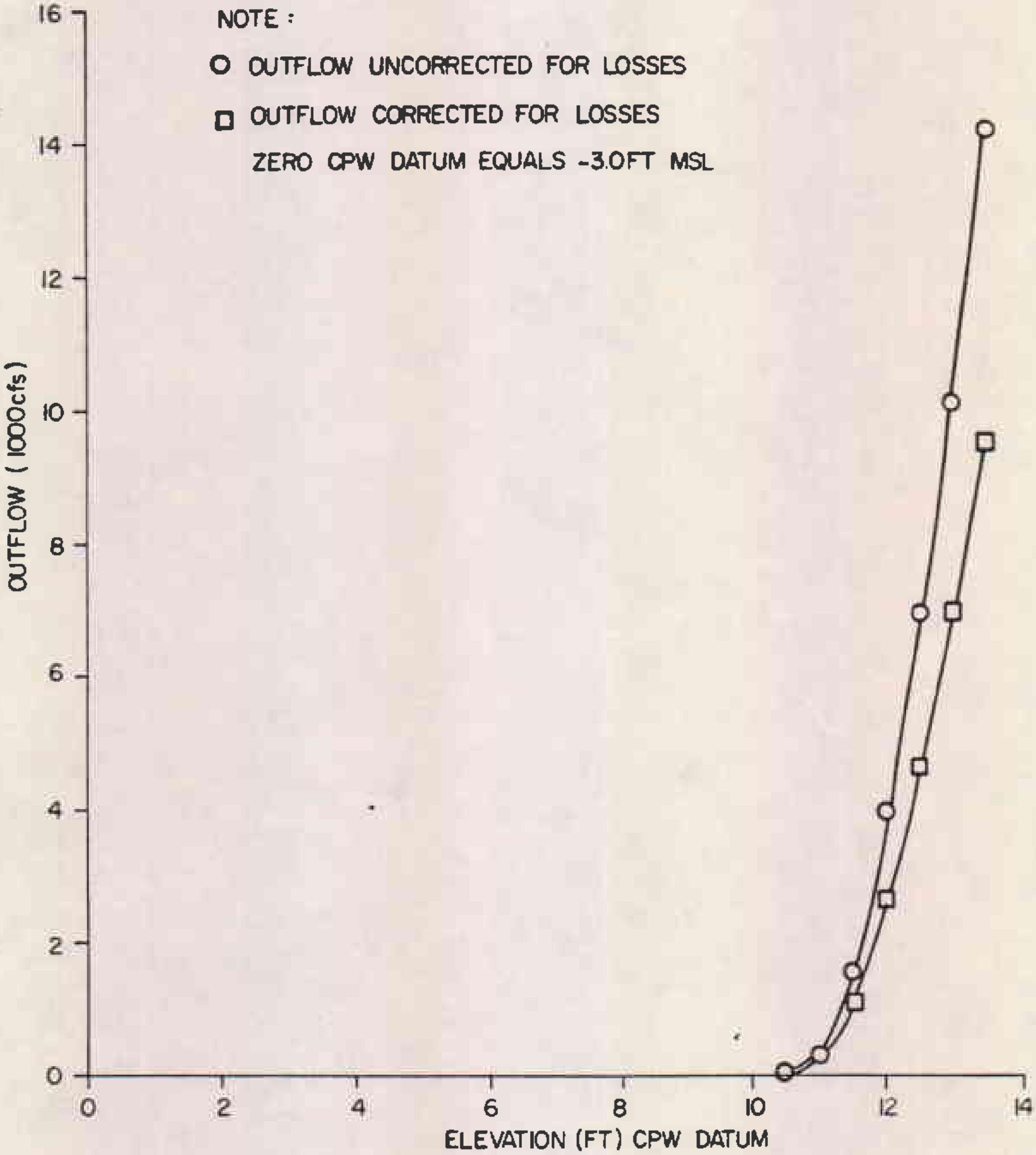


Figure 12

GOOSE CREEK RESERVOIR OUTFLOW VERSUS ELEVATION

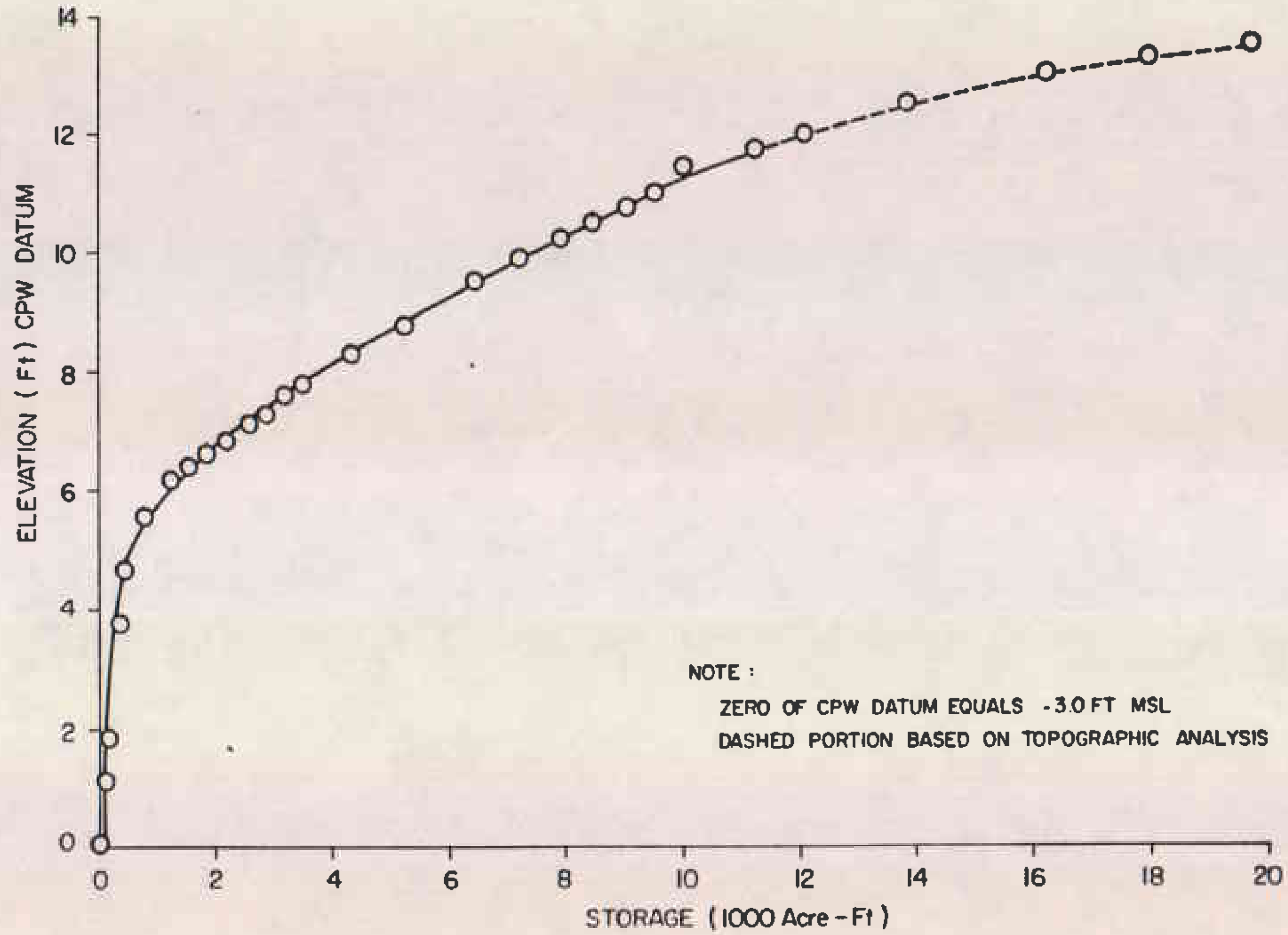


Figure 13
GOOSE CREEK RESERVOIR STORAGE VERSUS ELEVATION

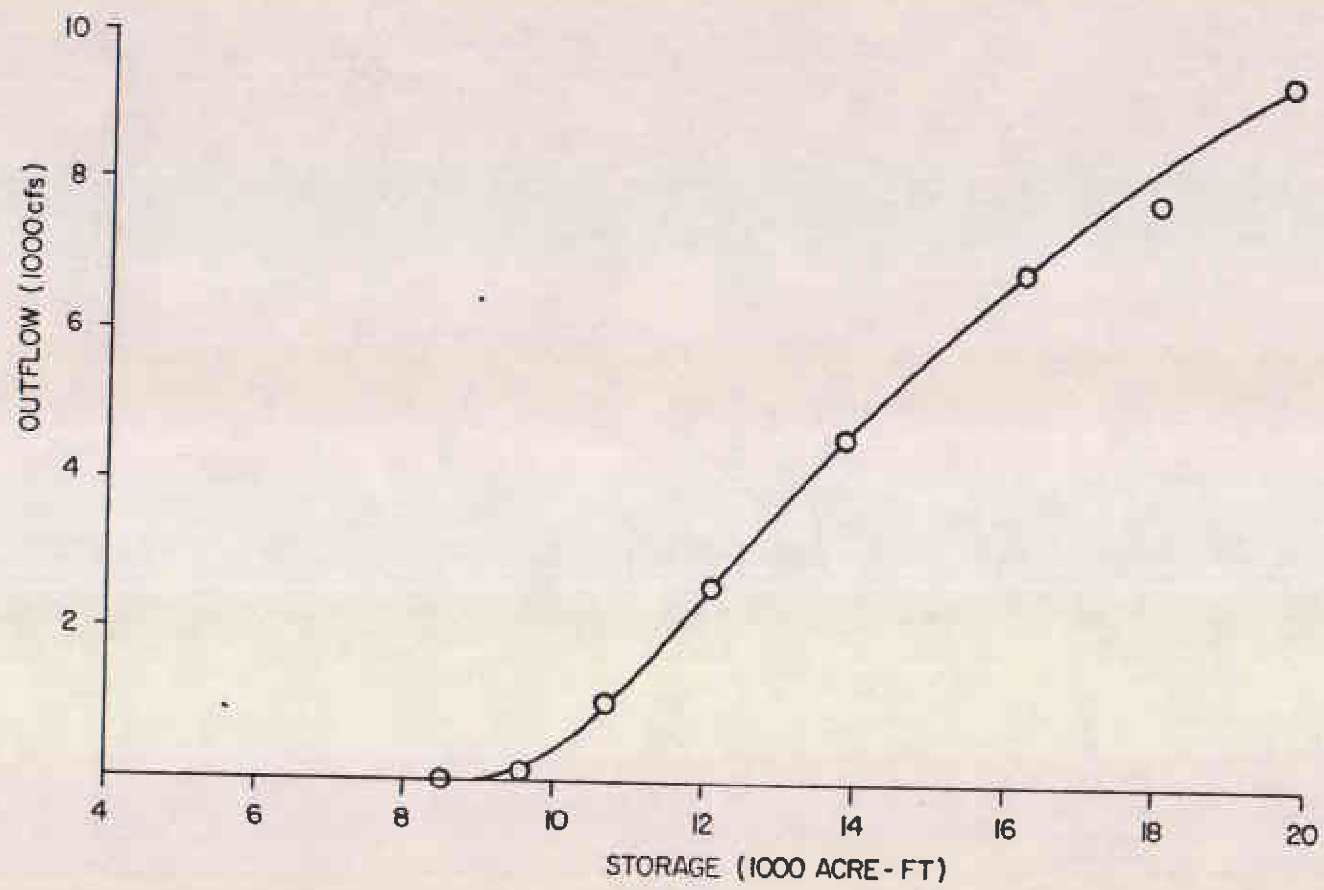


Figure 14
GOOSE CREEK RESERVOIR STORAGE VERSUS OUTFLOW

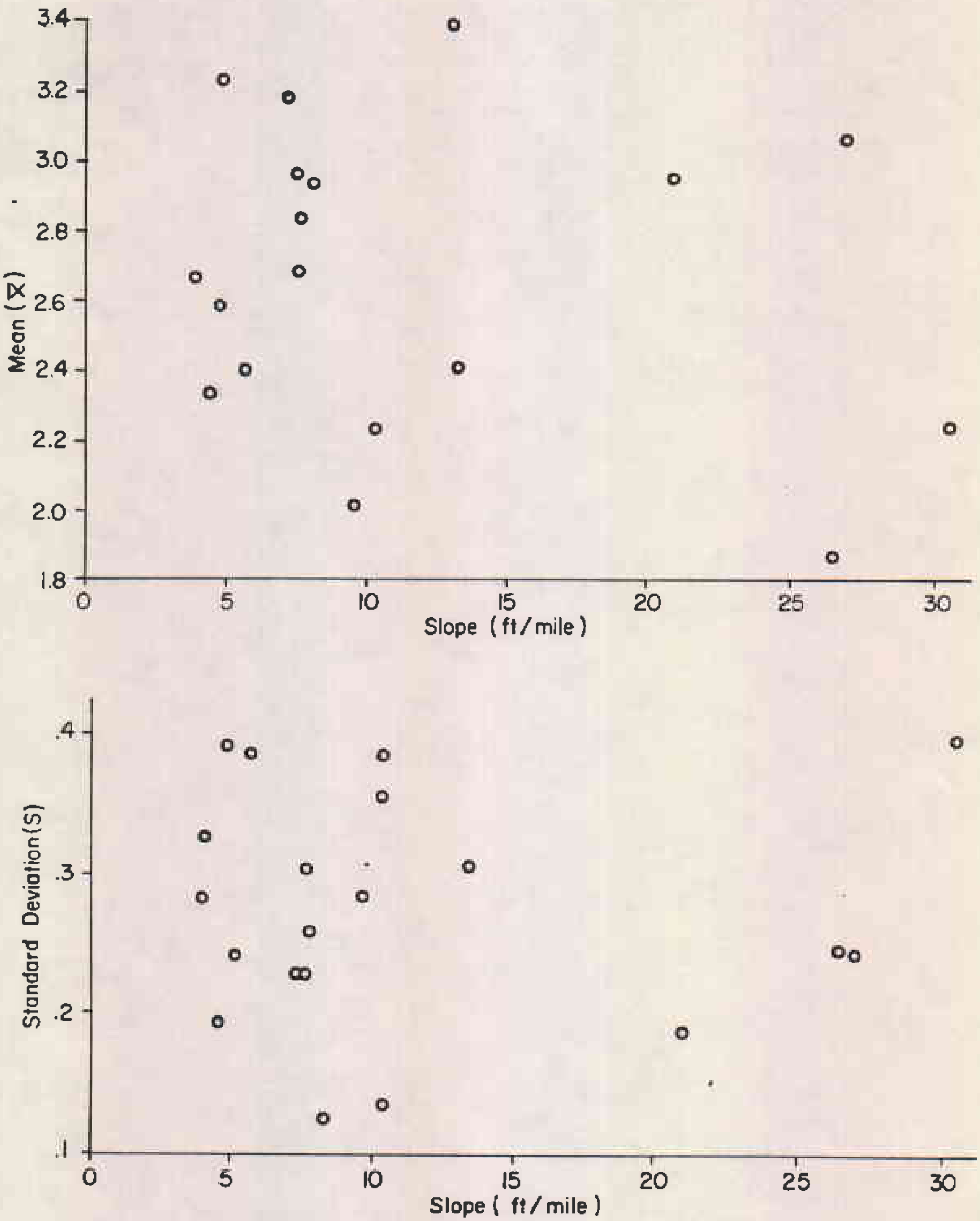


Figure 15

MEAN AND STANDARD DEVIATION OF FLOOD FREQUENCY DISTRIBUTION VERSUS MAINSTREAM SLOPE

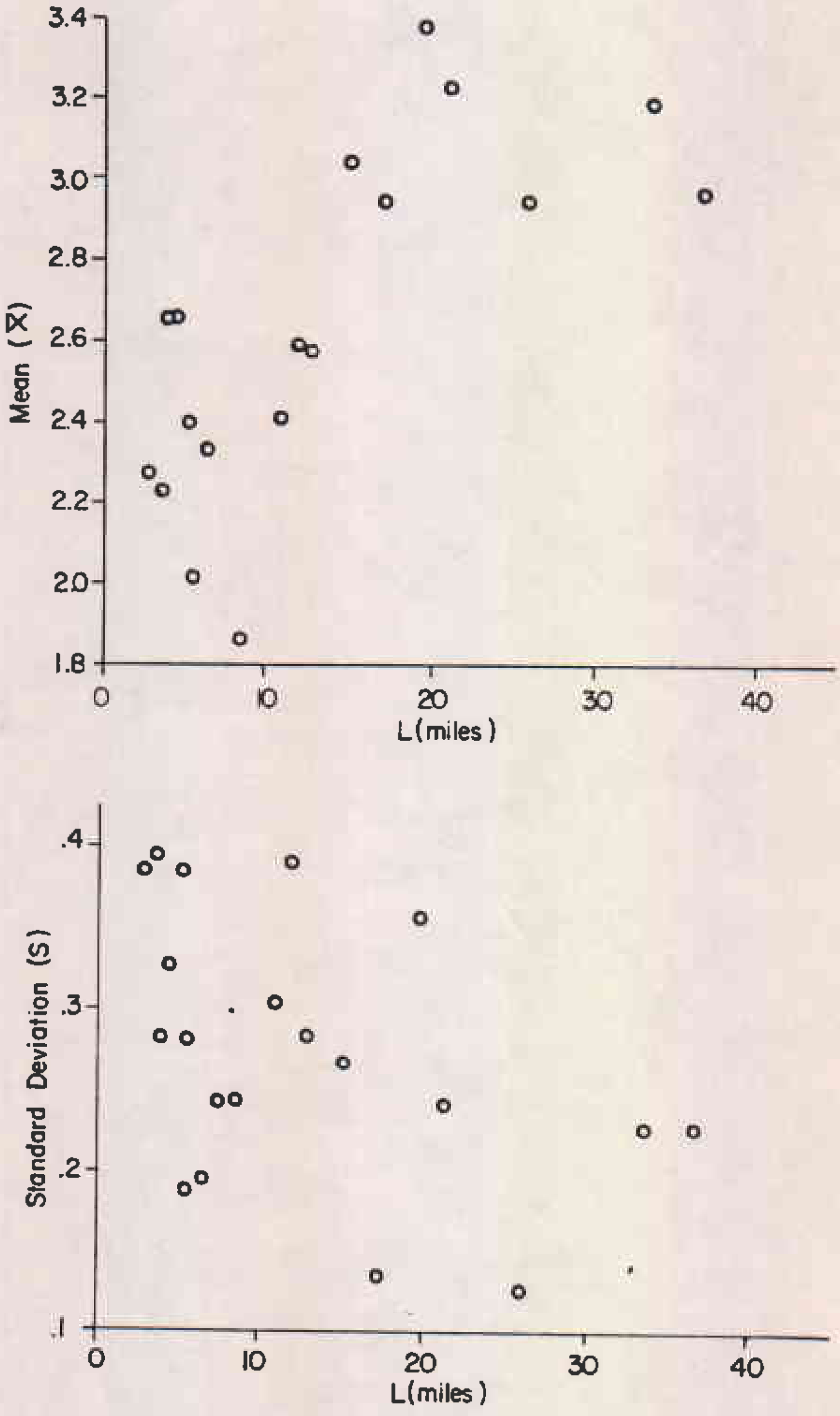


Figure 16

MEAN AND STANDARD DEVIATION OF FLOOD FREQUENCY DISTRIBUTION VERSUS MAINSTREAM LENGTH

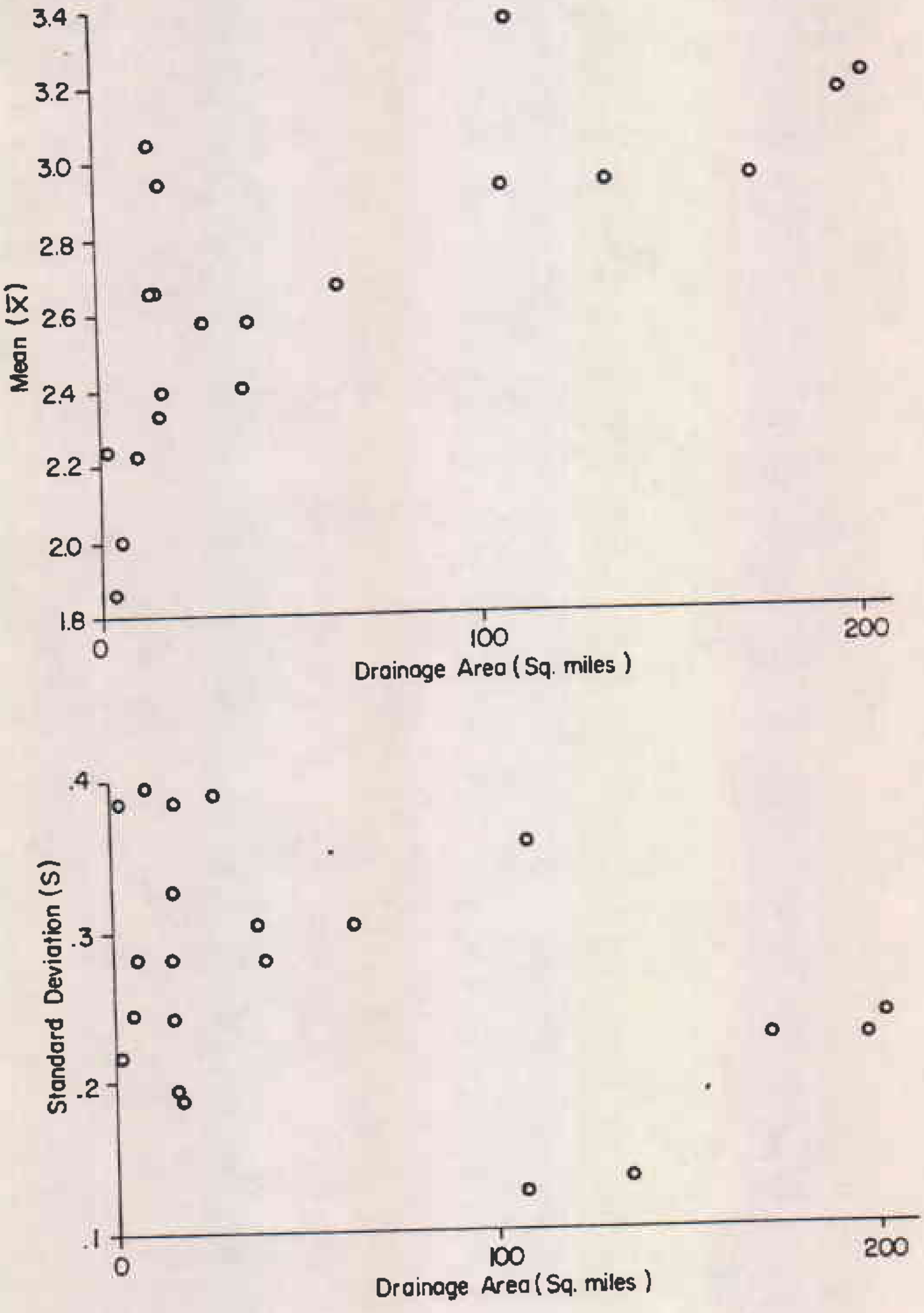


Figure 17

MEAN AND STANDARD DEVIATION OF FLOOD FREQUENCY DISTRIBUTION VERSUS DRAINAGE AREA

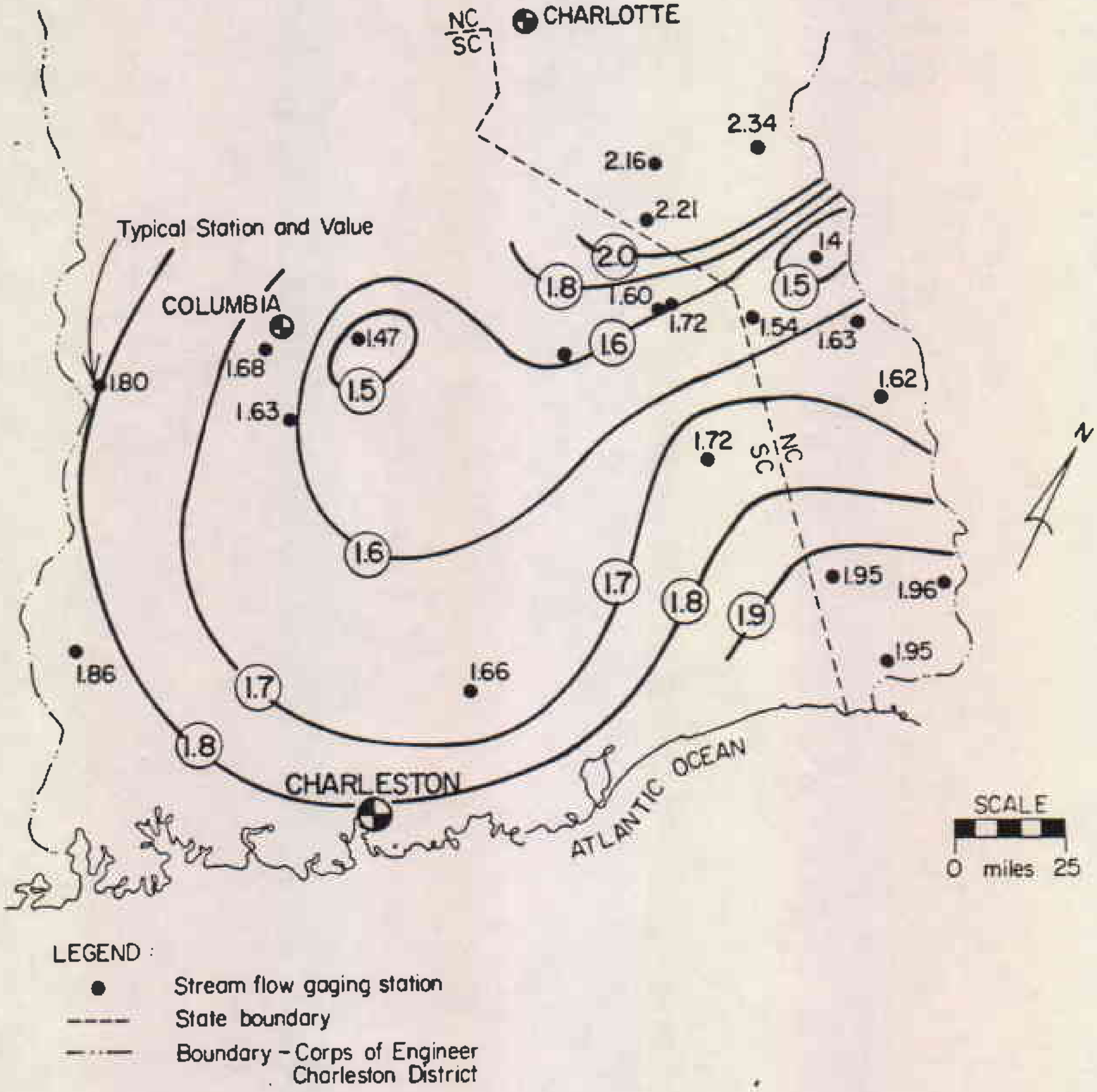


Figure 18

GEOGRAPHICAL VARIATION OF COEFFICIENT, CM, FOR MEAN REGRESSION OF ANNUAL FLOOD PEAKS

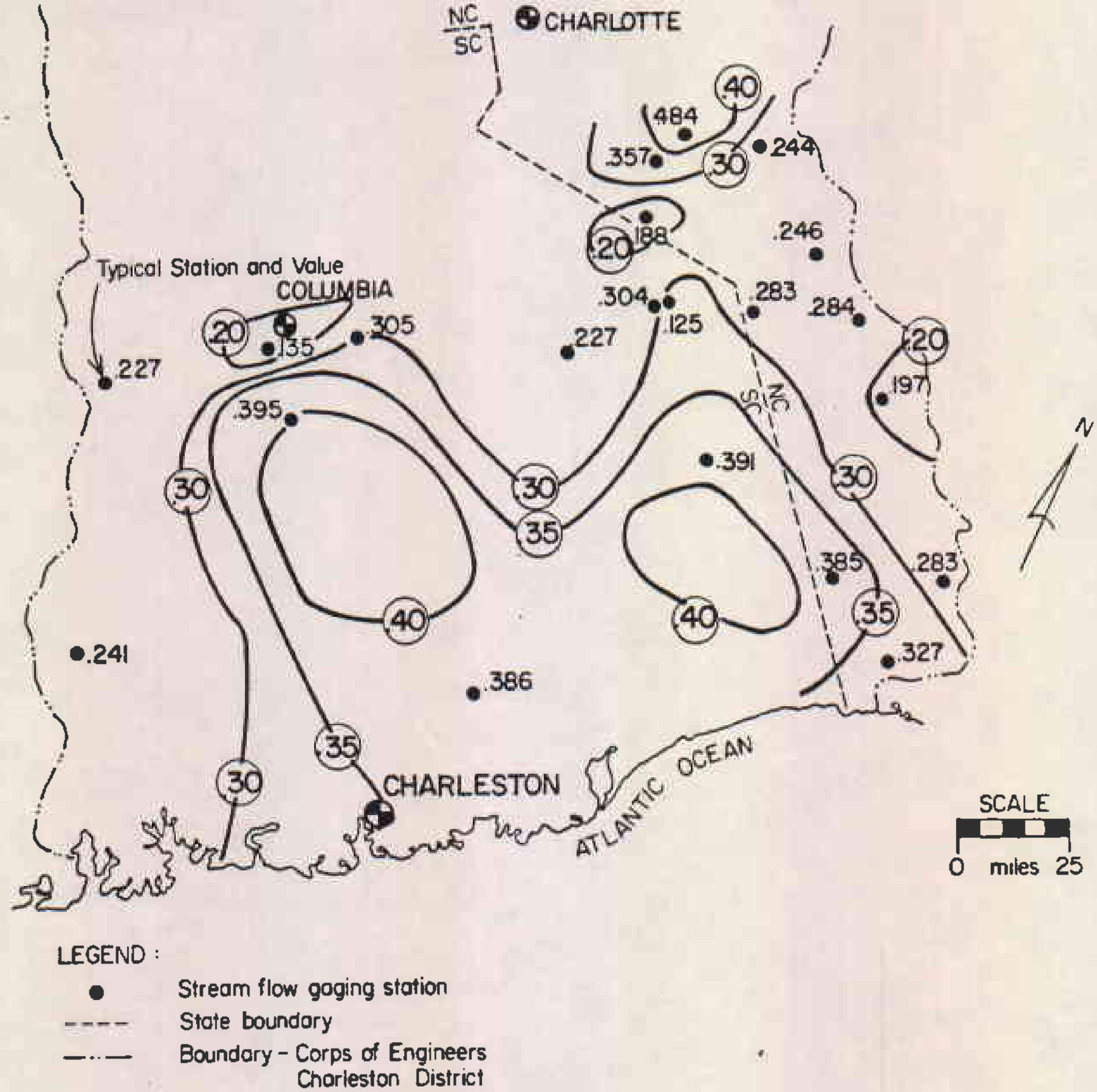


Figure 19

GEOGRAPHICAL VARIATION OF THE STANDARD DEVIATIONS OF ANNUAL FLOOD PEAKS

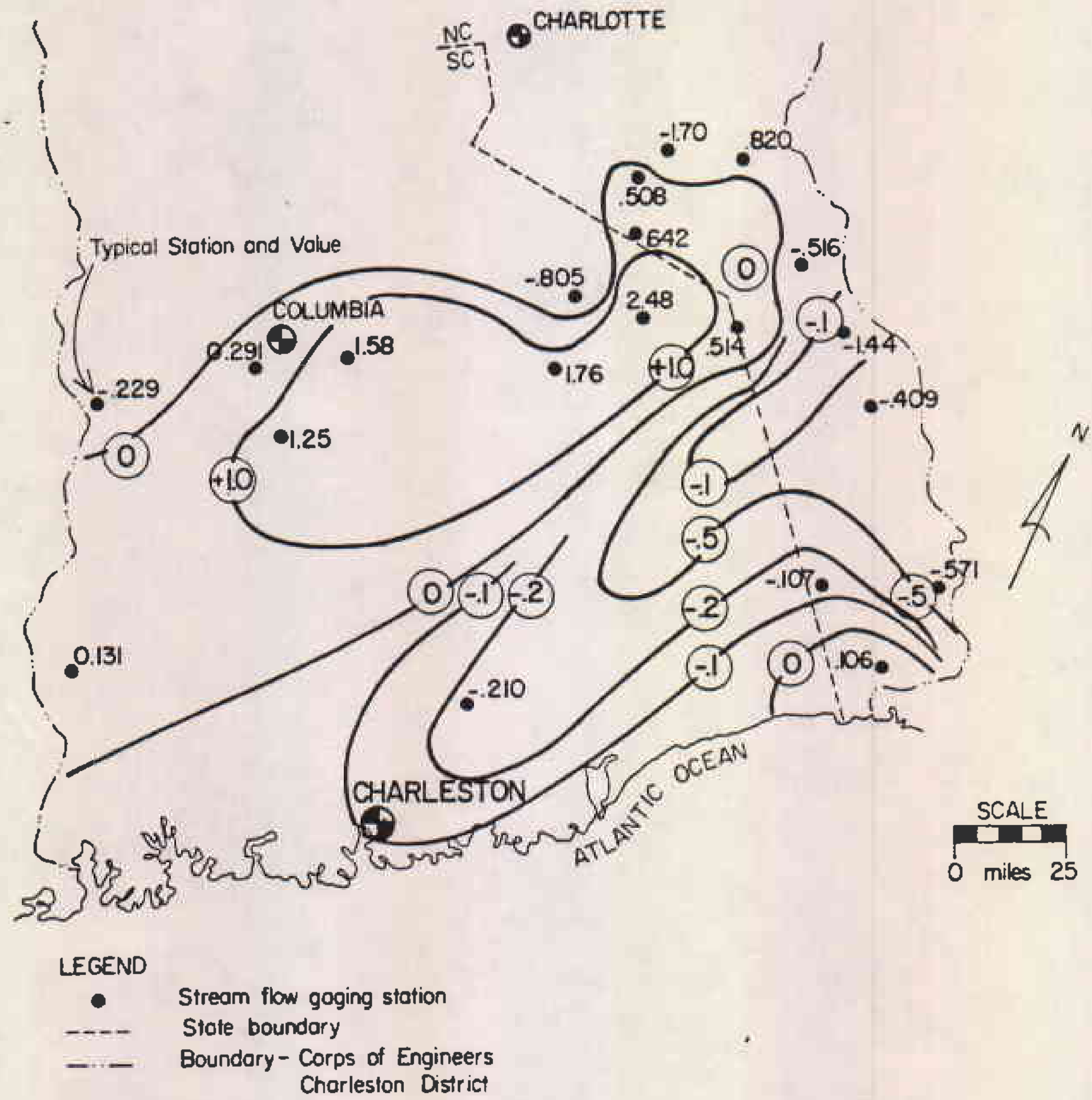


Figure 20

GEOGRAPHICAL VARIATION OF SKEW COEFFICIENTS OF ANNUAL FLOOD PEAKS

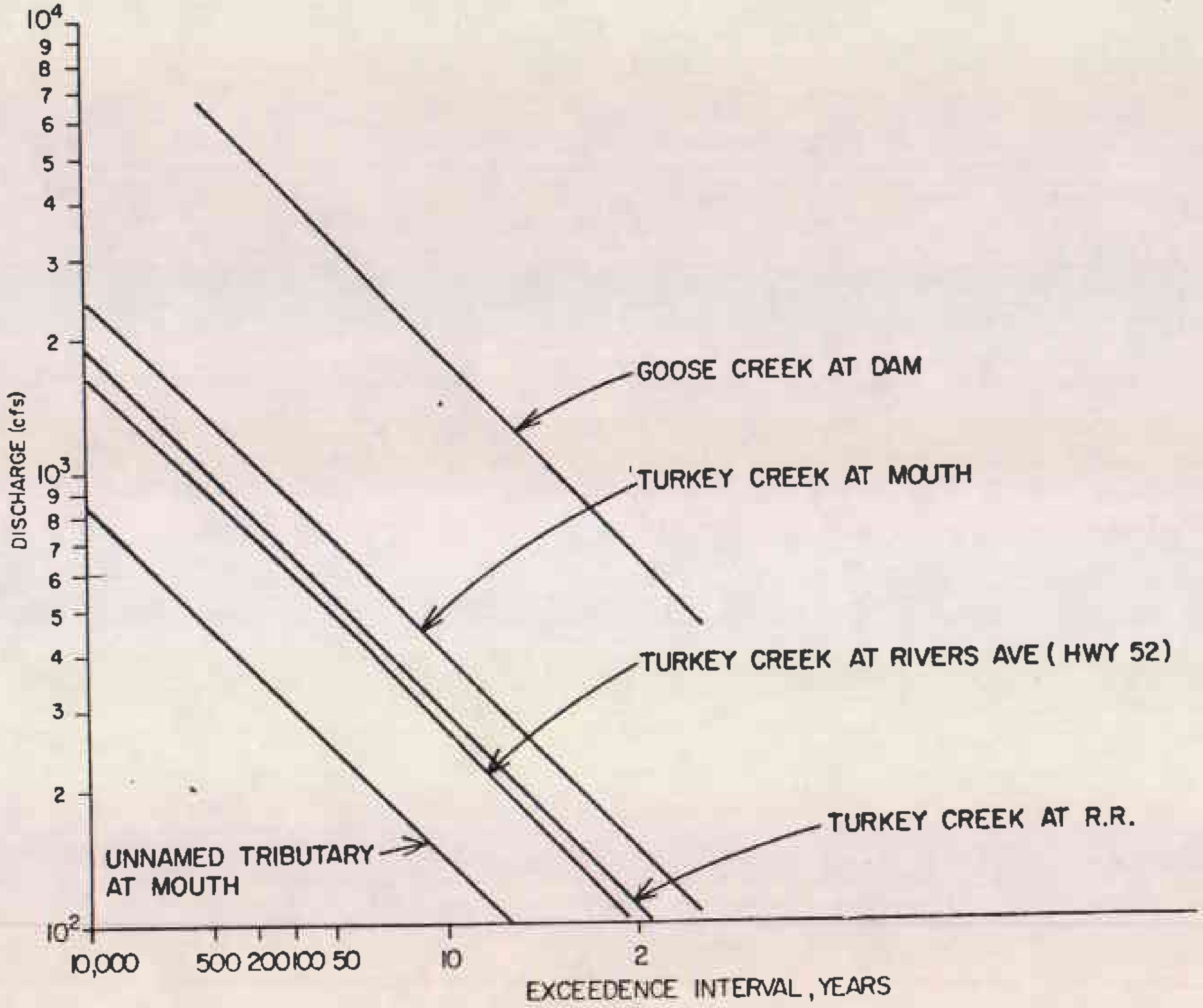


Figure 21

ANNUAL FLOOD PEAK FREQUENCY CURVES FROM REGIONAL ANALYSIS
(Not corrected for urbanization)

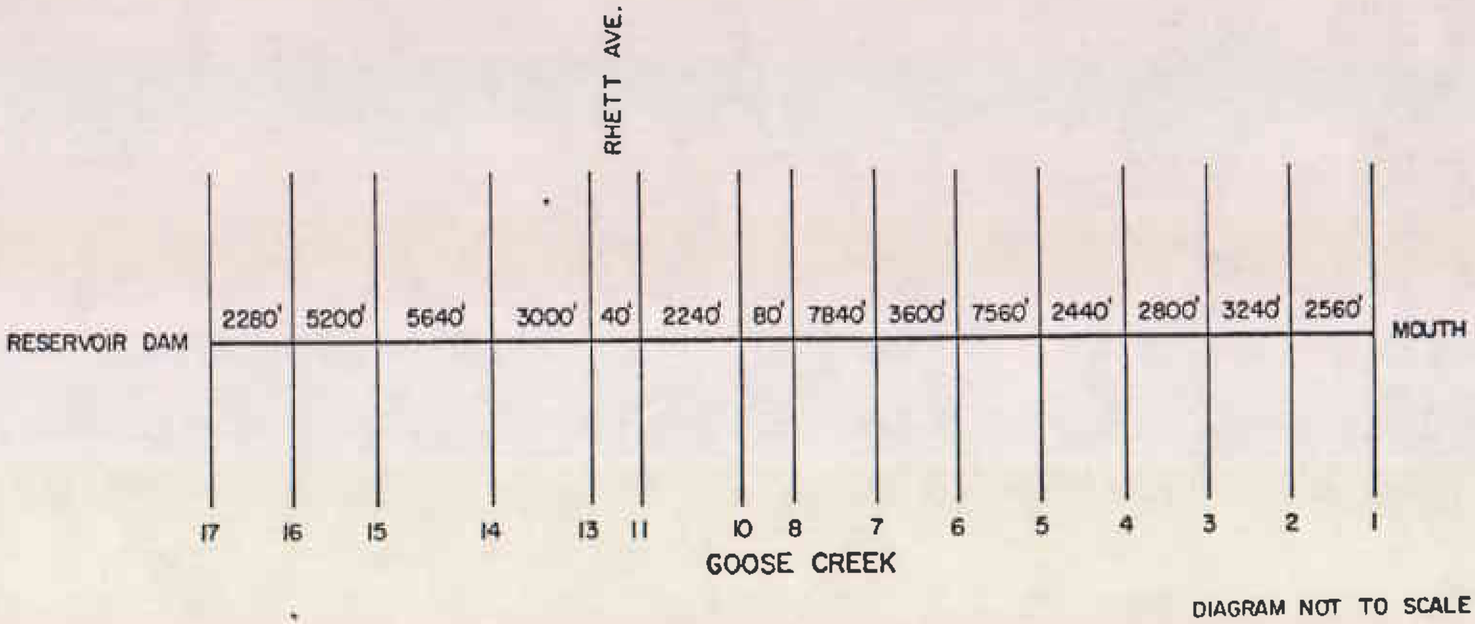


Figure 22
HEC-2 MODEL CROSS-SECTION LOCATIONS
(SHOWING CROSSINGS AND DISTANCES BETWEEN EACH SECTION IN CHANNEL)

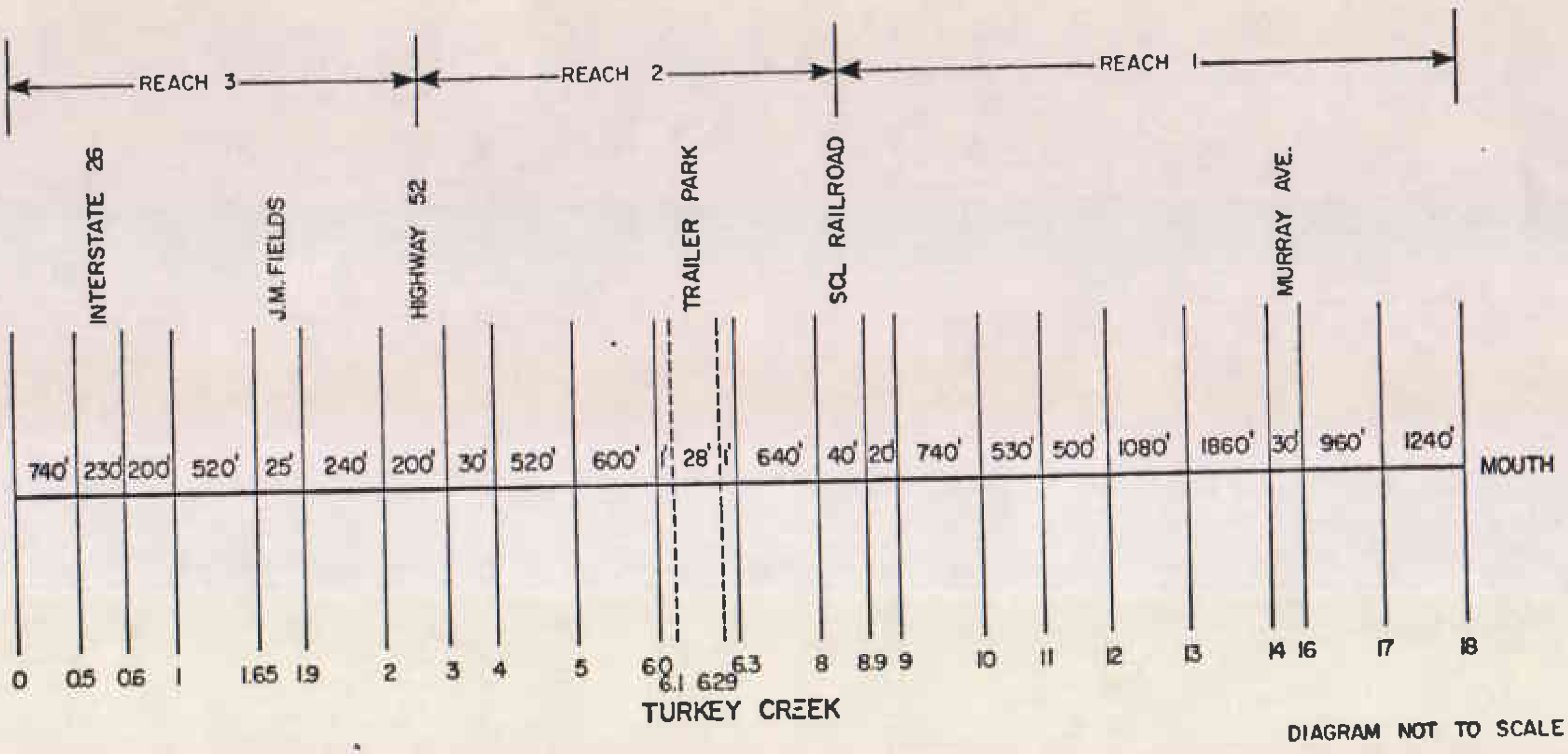


Figure 23

HEC-2 MODEL CROSS-SECTION LOCATIONS

(SHOWING CROSSINGS AND DISTANCES BETWEEN EACH SECTION IN CHANNEL)

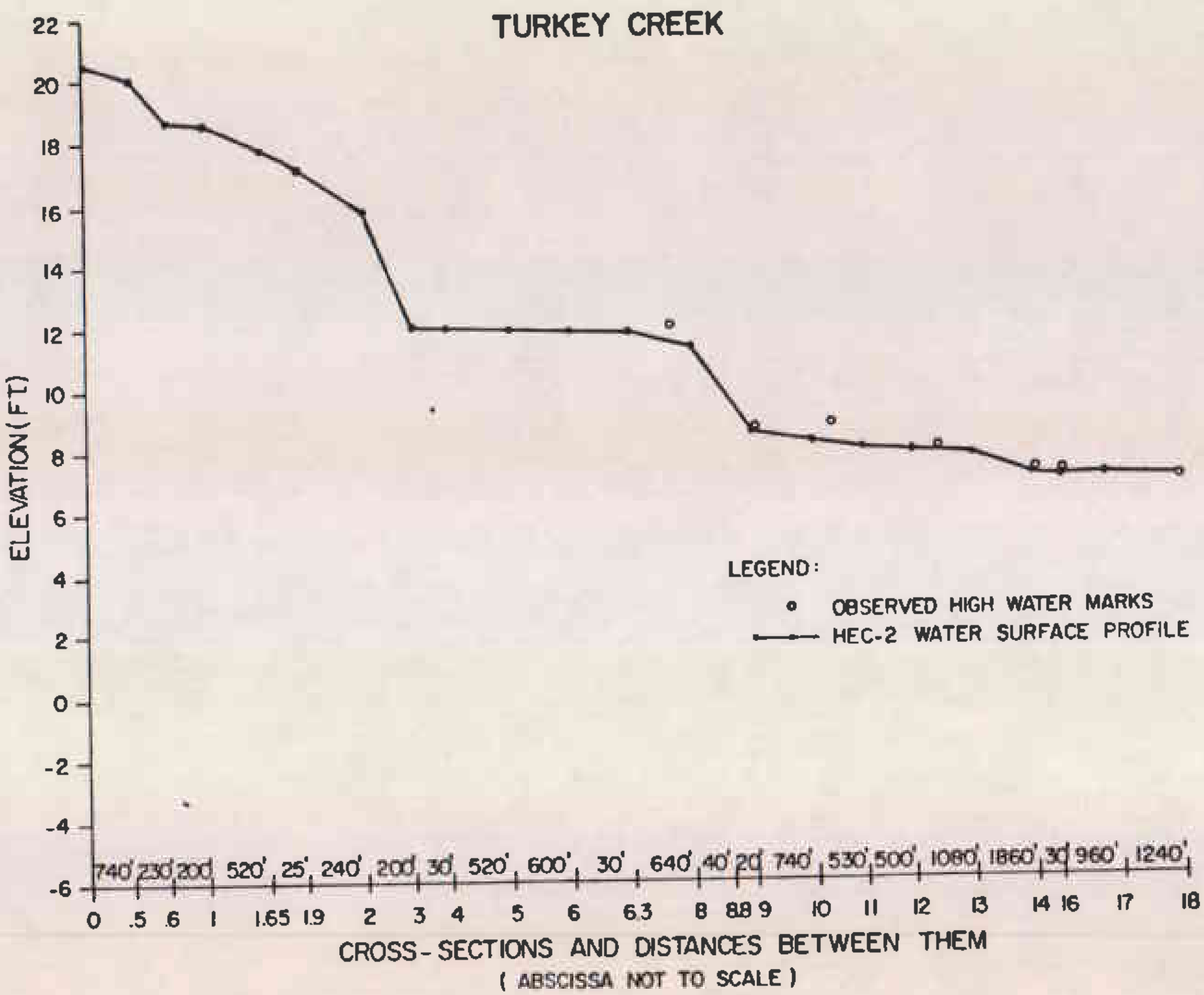


Figure 24

COMPUTED VS. OBSERVED WATER SURFACE ELEVATIONS FOR CALIBRATION PERIOD
JUNE 1973

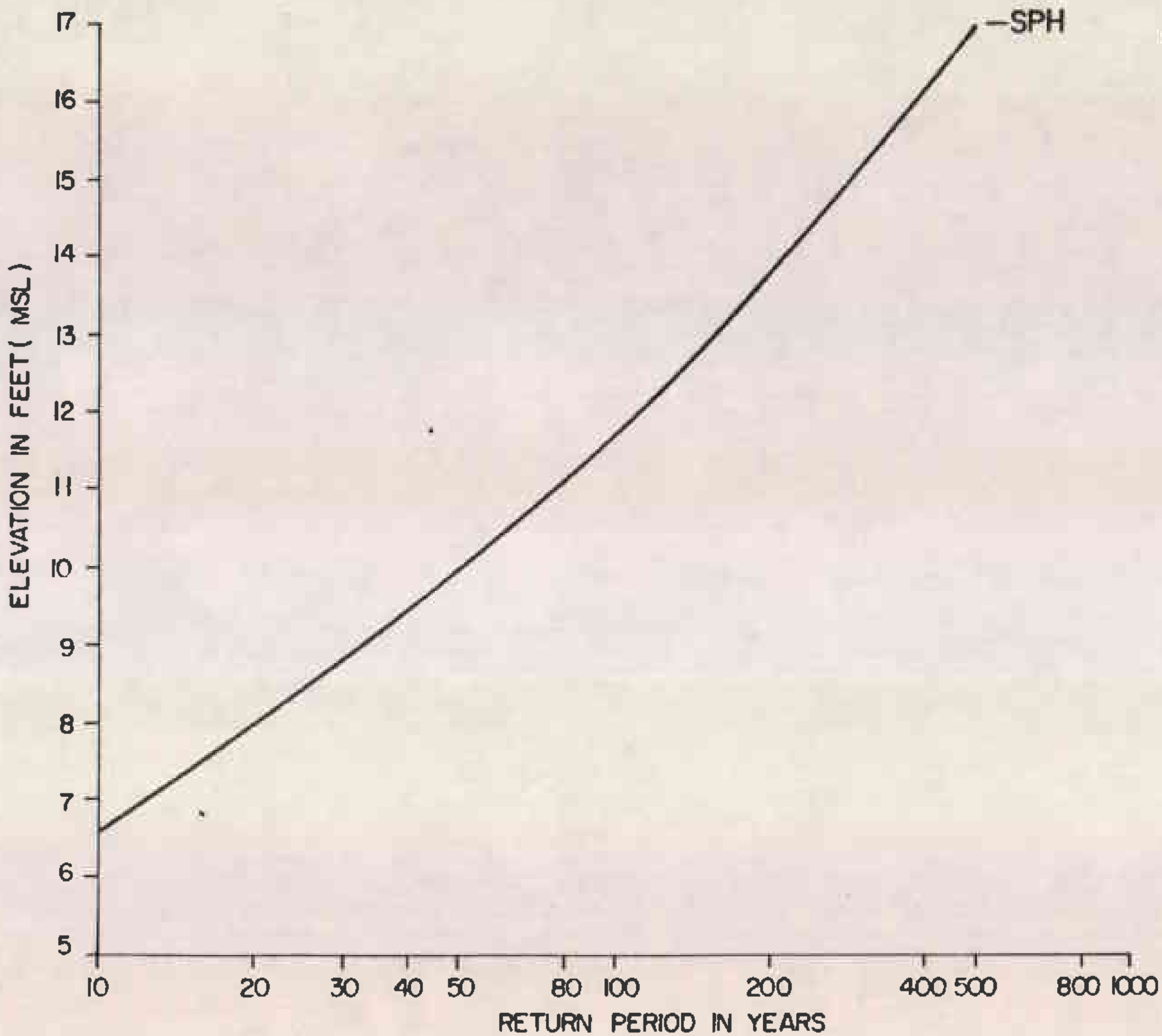


Figure 25

HURRICANE SURGE FREQUENCIES ON COOPER RIVER IN THE VICINITY OF GOOSE CREEK

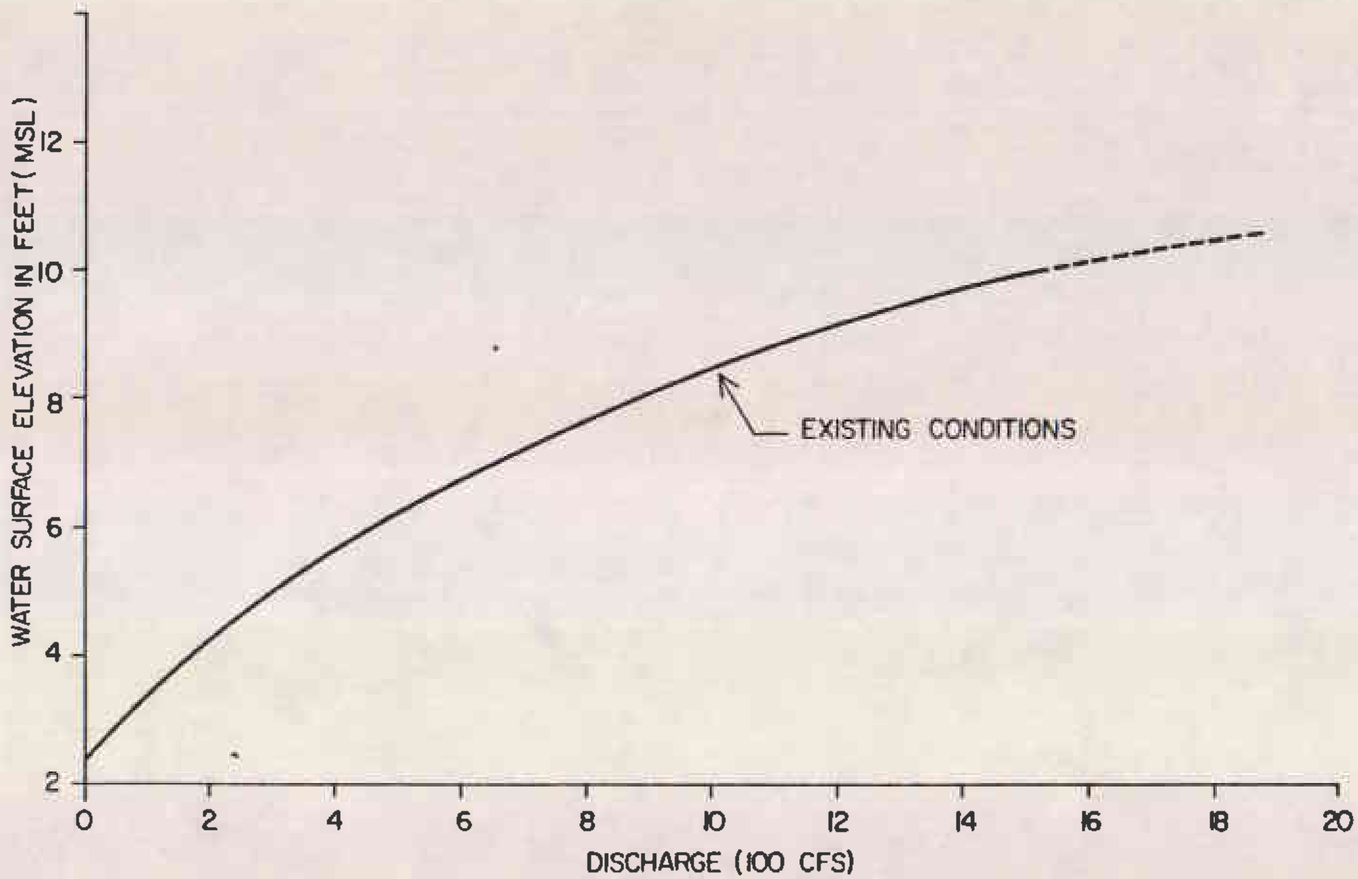


Figure 26
TURKEY CREEK STAGE-DISCHARGE, REACH I

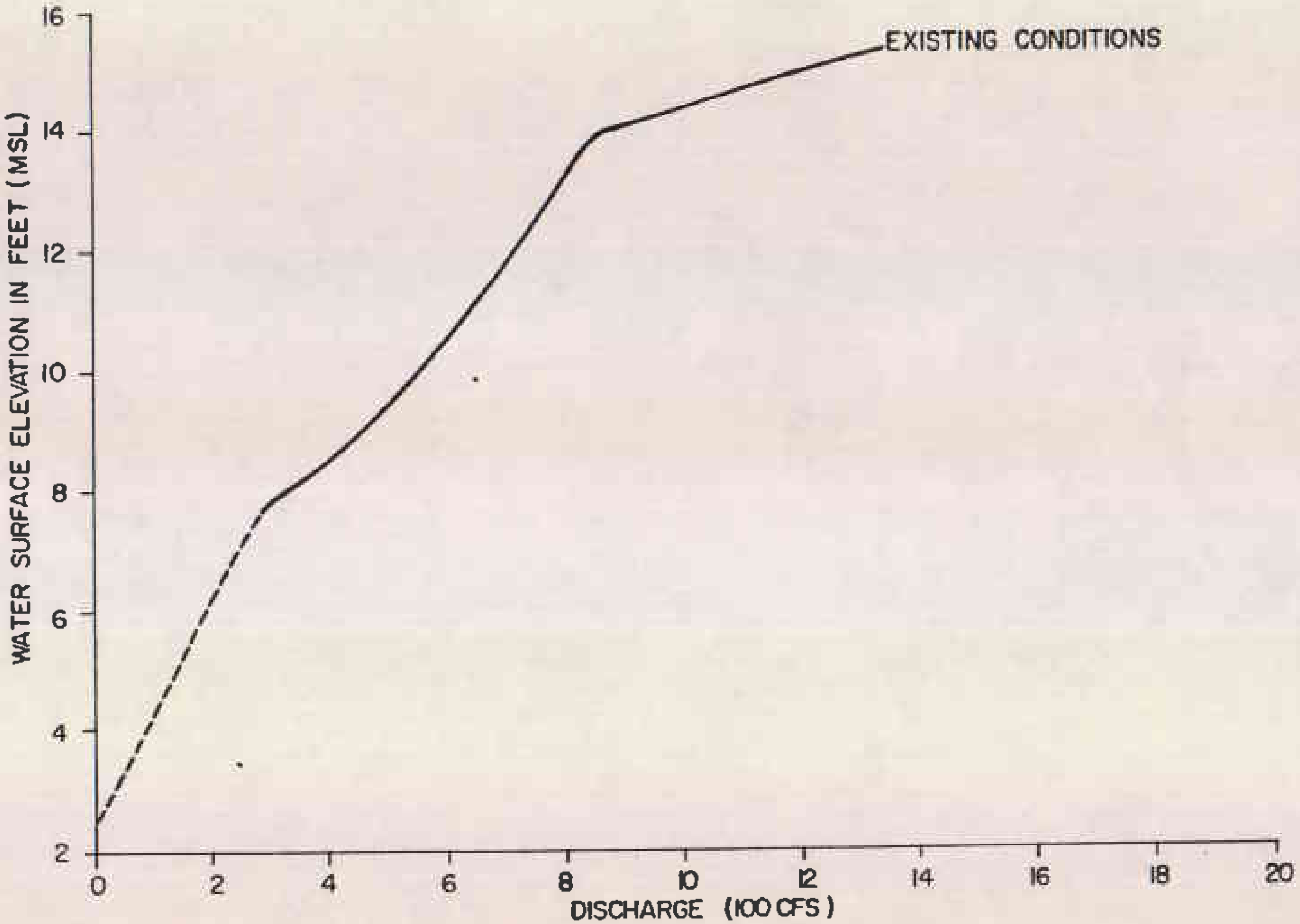


Figure 27

TURKEY CREEK STAGE-DISCHARGE , REACH 2

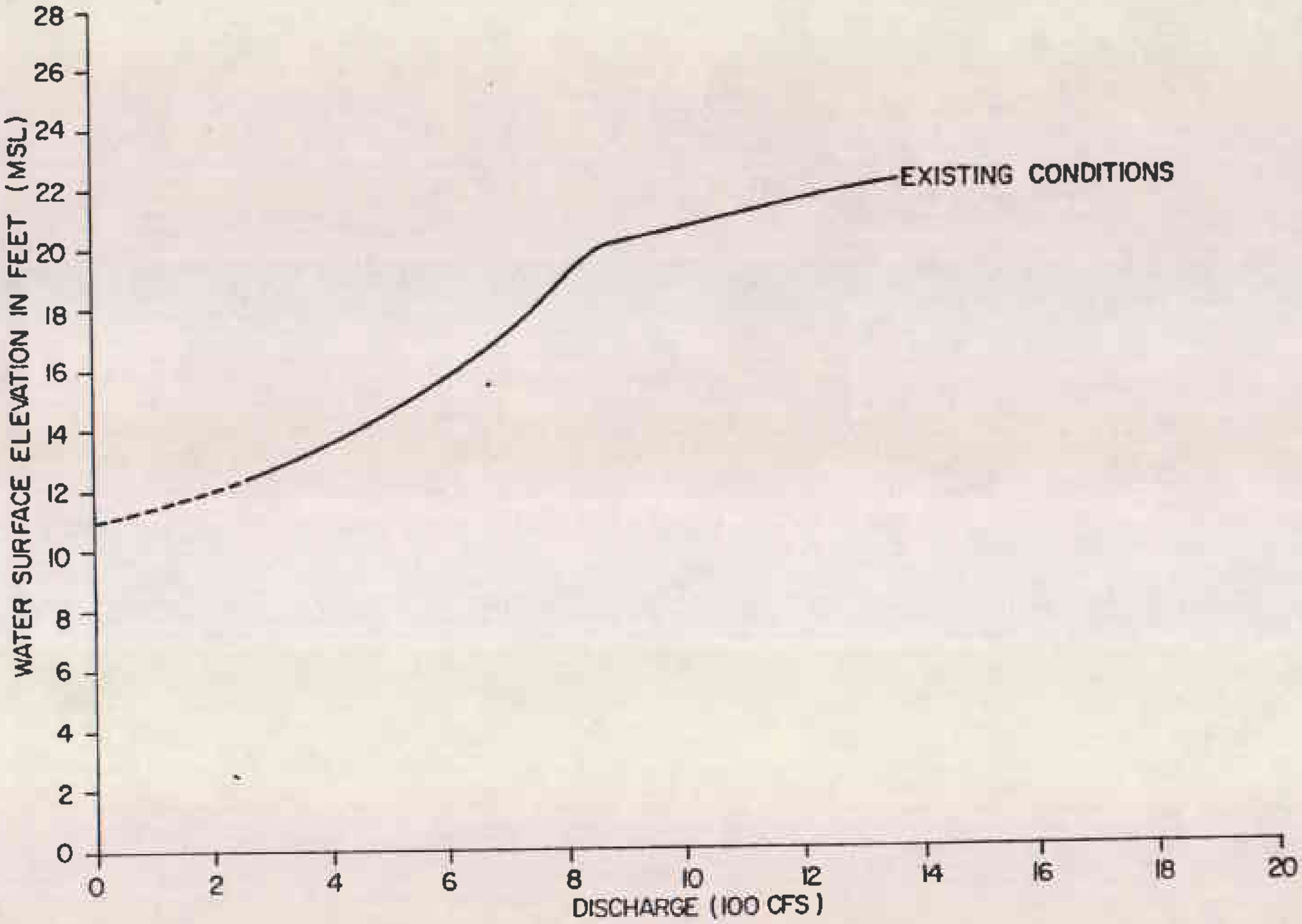


Figure 28

TURKEY CREEK STAGE-DISCHARGE , REACH 3

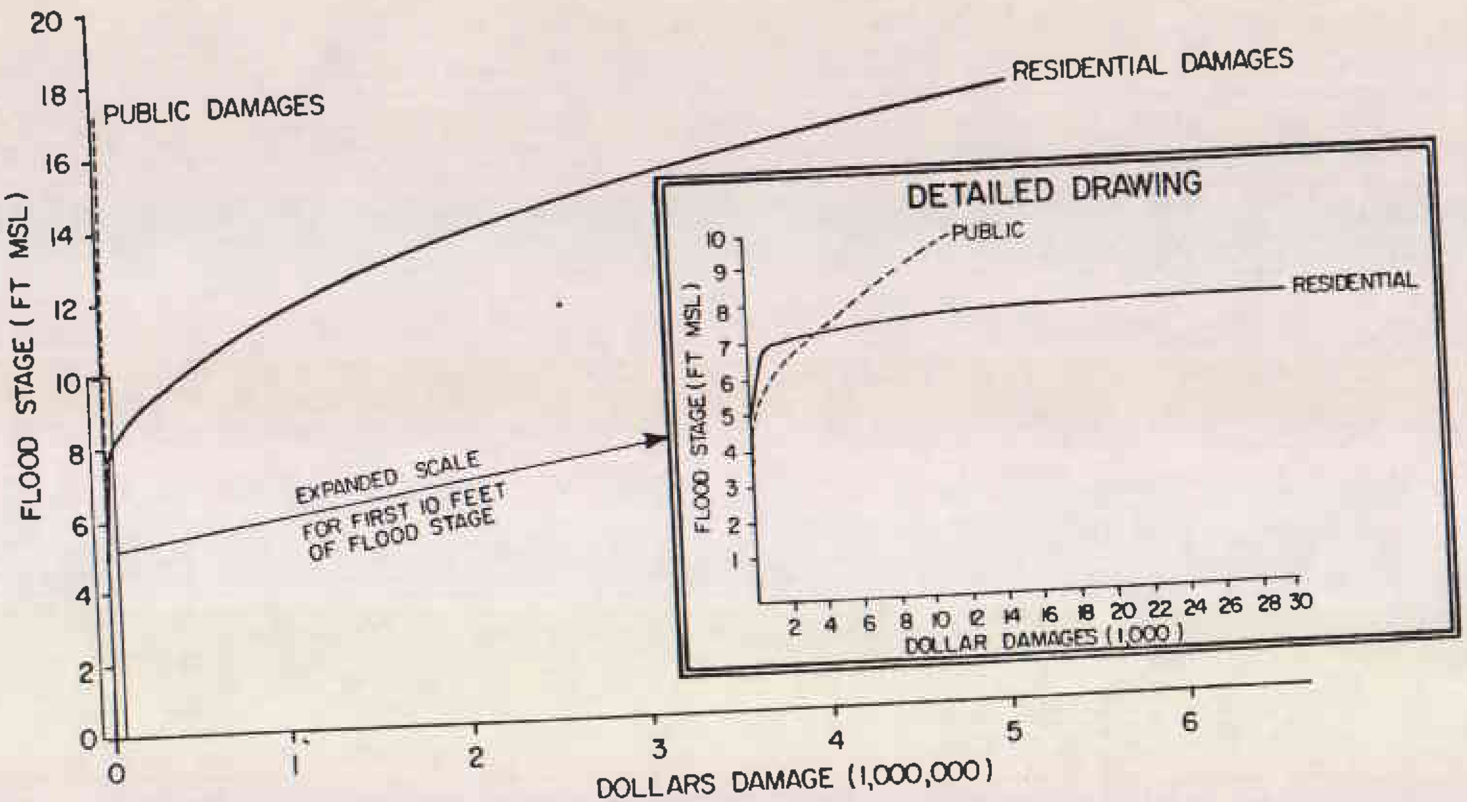


Figure 29

TURKEY CREEK STAGE DAMAGES , REACH I

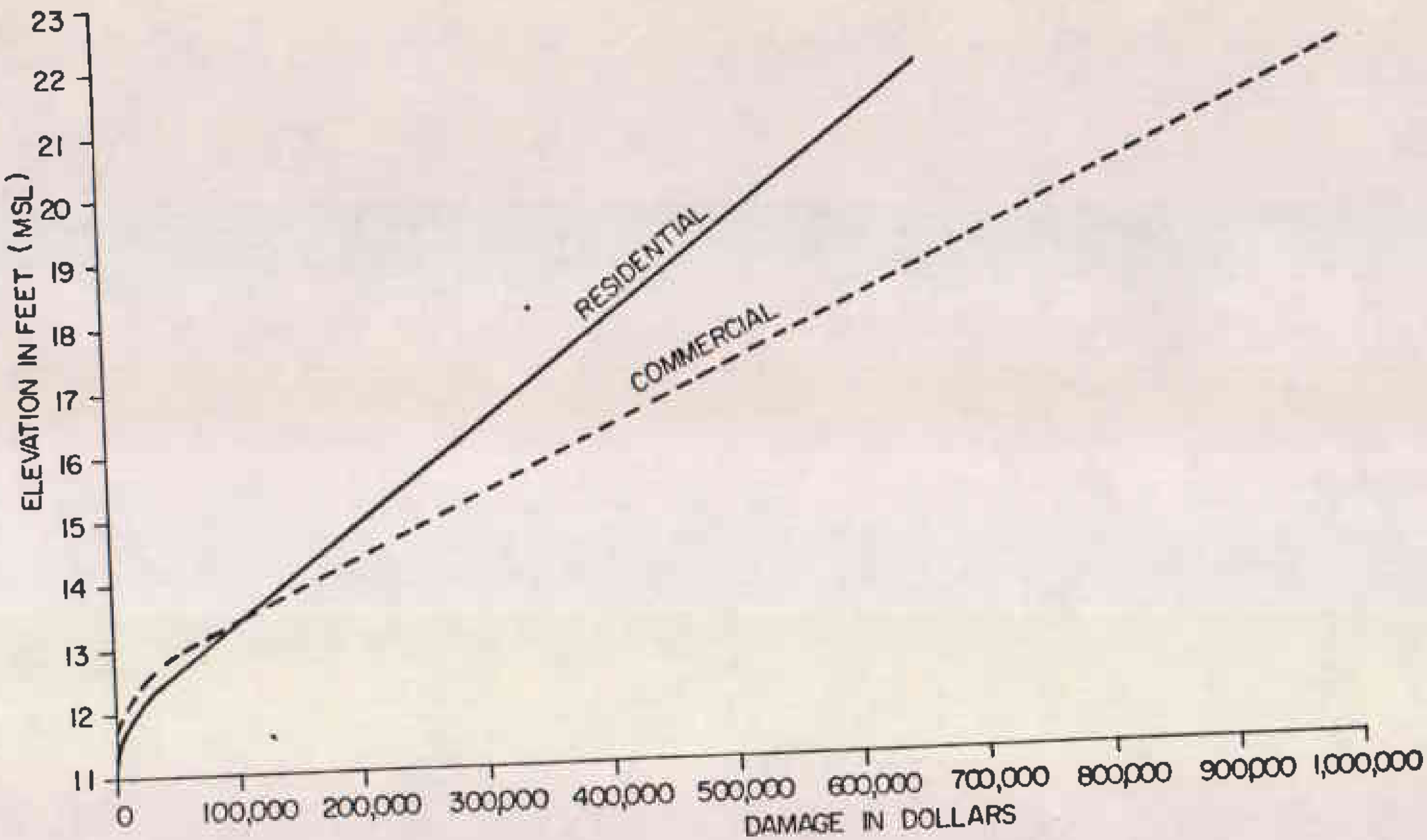


Figure 30

TURKEY CREEK STAGE DAMAGE, REACH 2

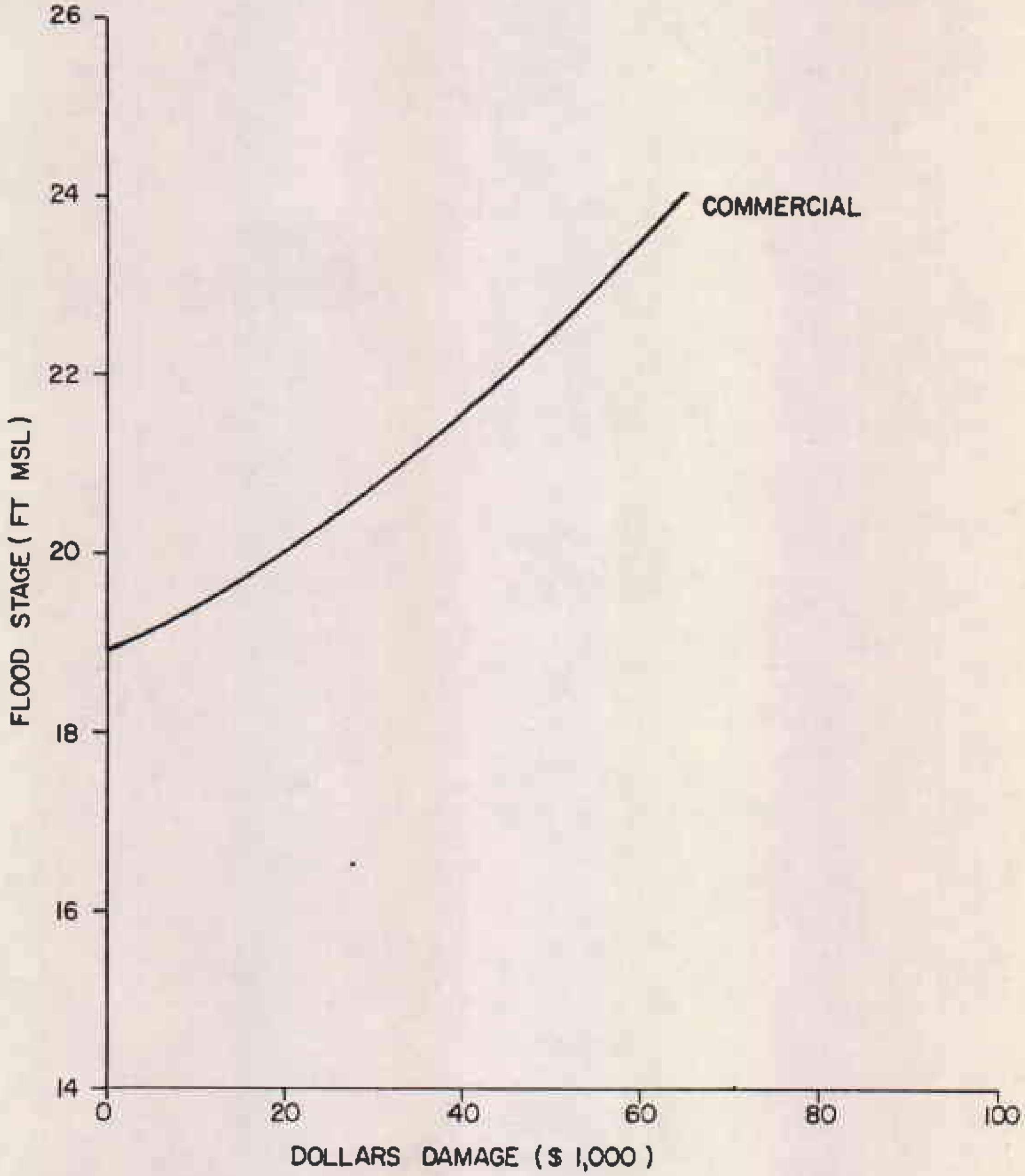


Figure 31

TURKEY CREEK STAGE DAMAGE , REACH 3

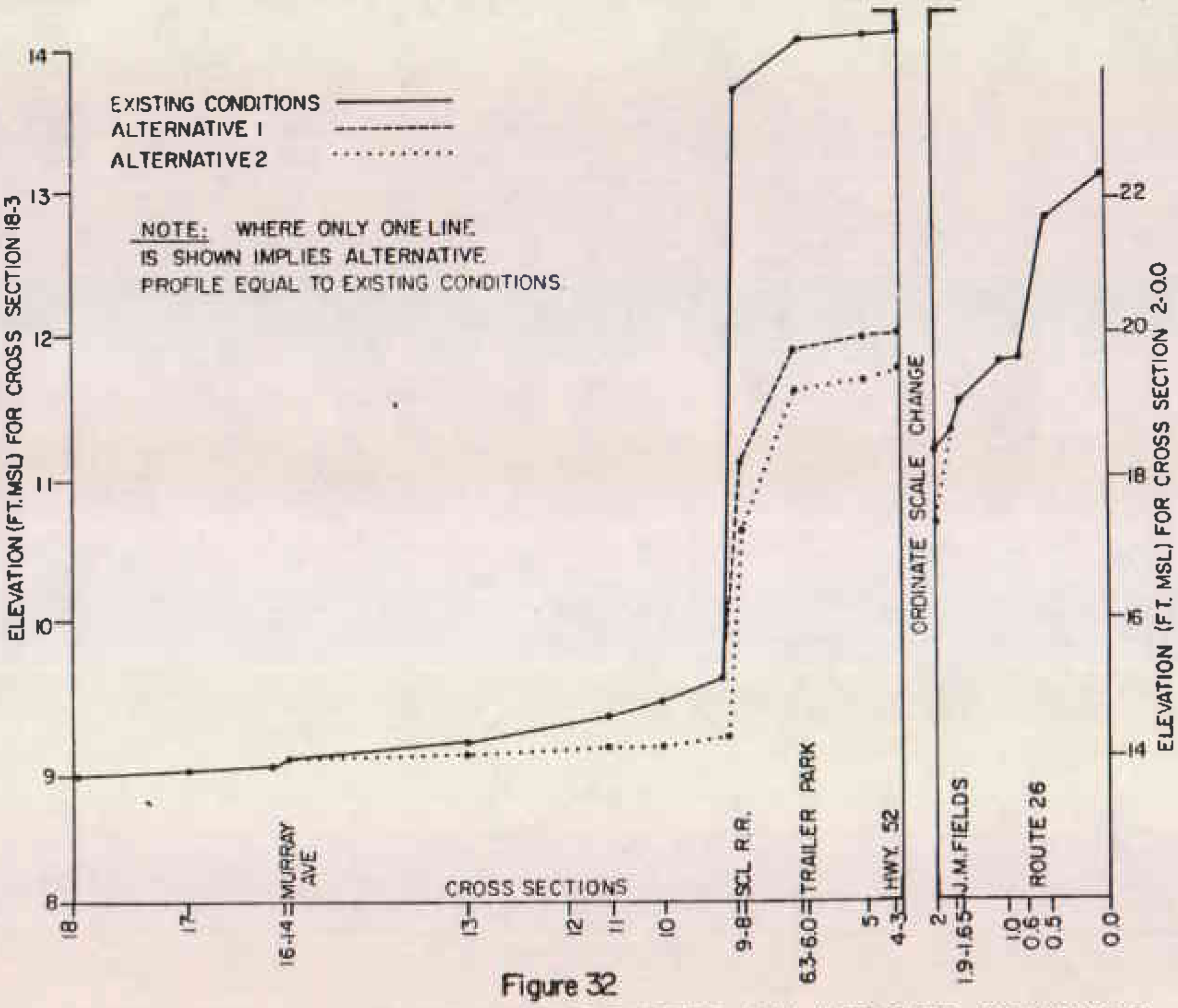


Figure 32

TURKEY CREEK 100 YR FLOOD PROFILE EXISTING AND IMPROVED CONDITIONS

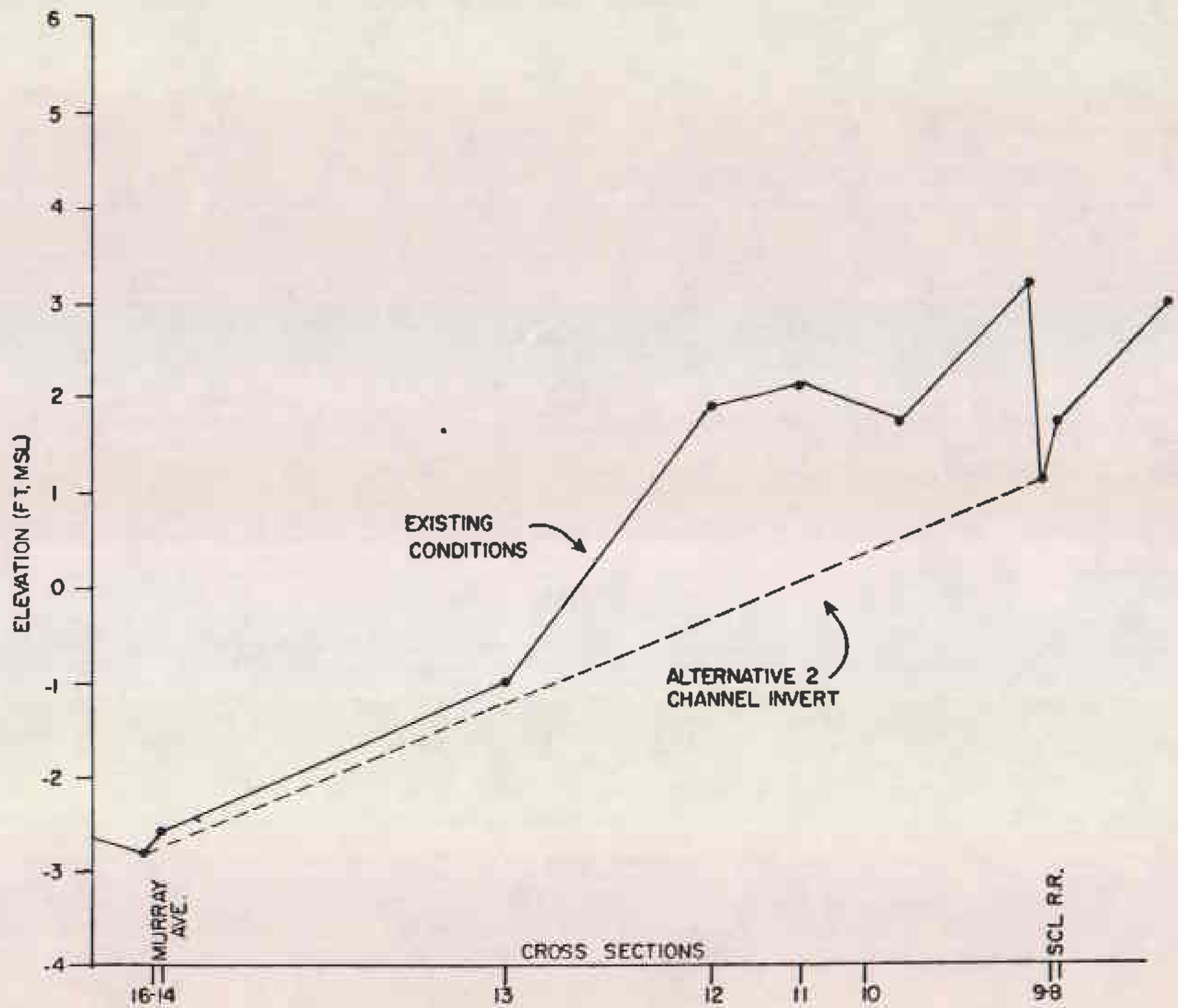


Figure 33

CHANNEL INVERT FOR ALTERNATIVE 2 TURKEY CREEK BETWEEN MURRAY AVENUE AND SCL RAILROAD

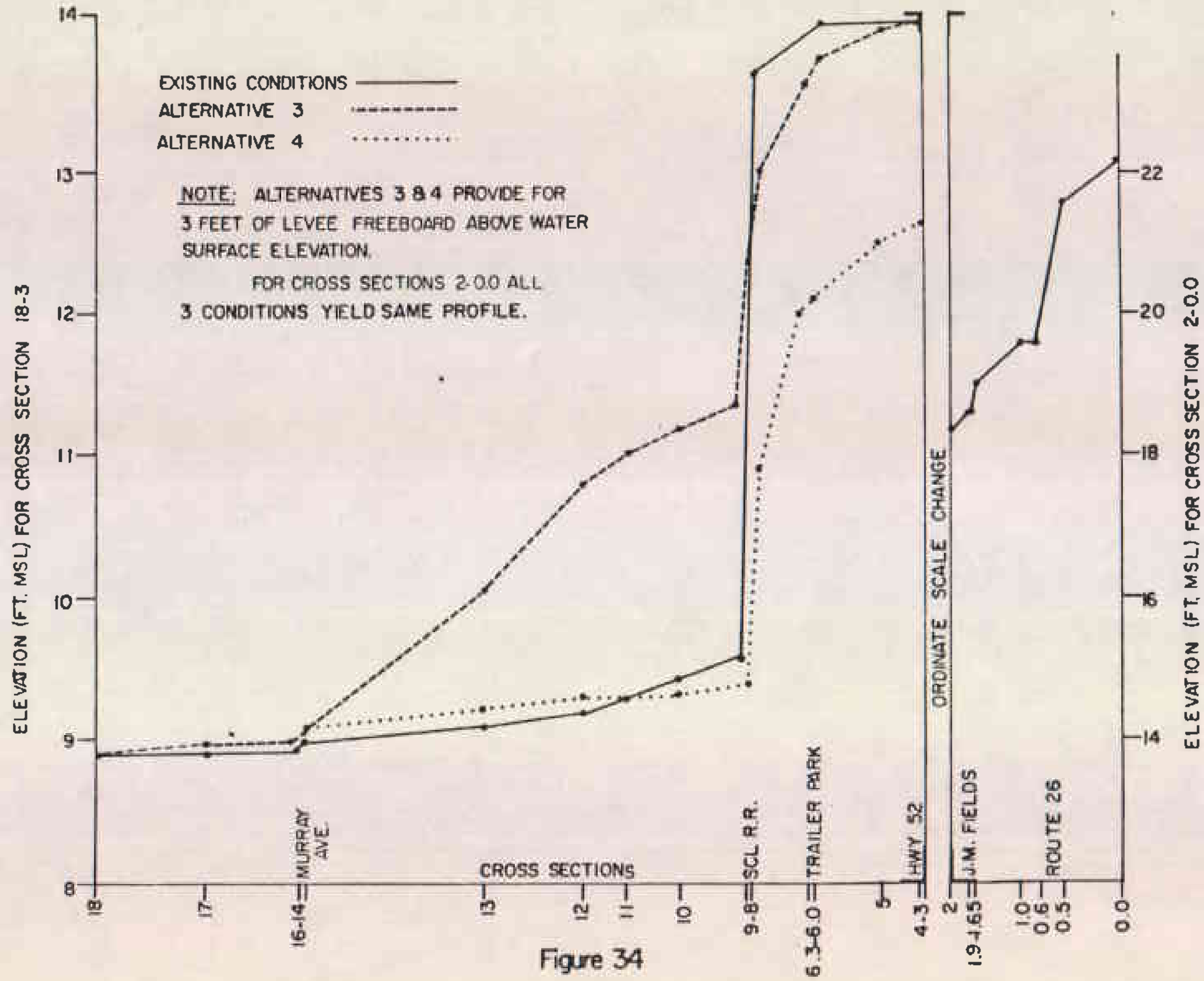


Figure 34

TURKEY CREEK 100YR FLOOD PROFILE EXISTING AND IMPROVED CONDITIONS

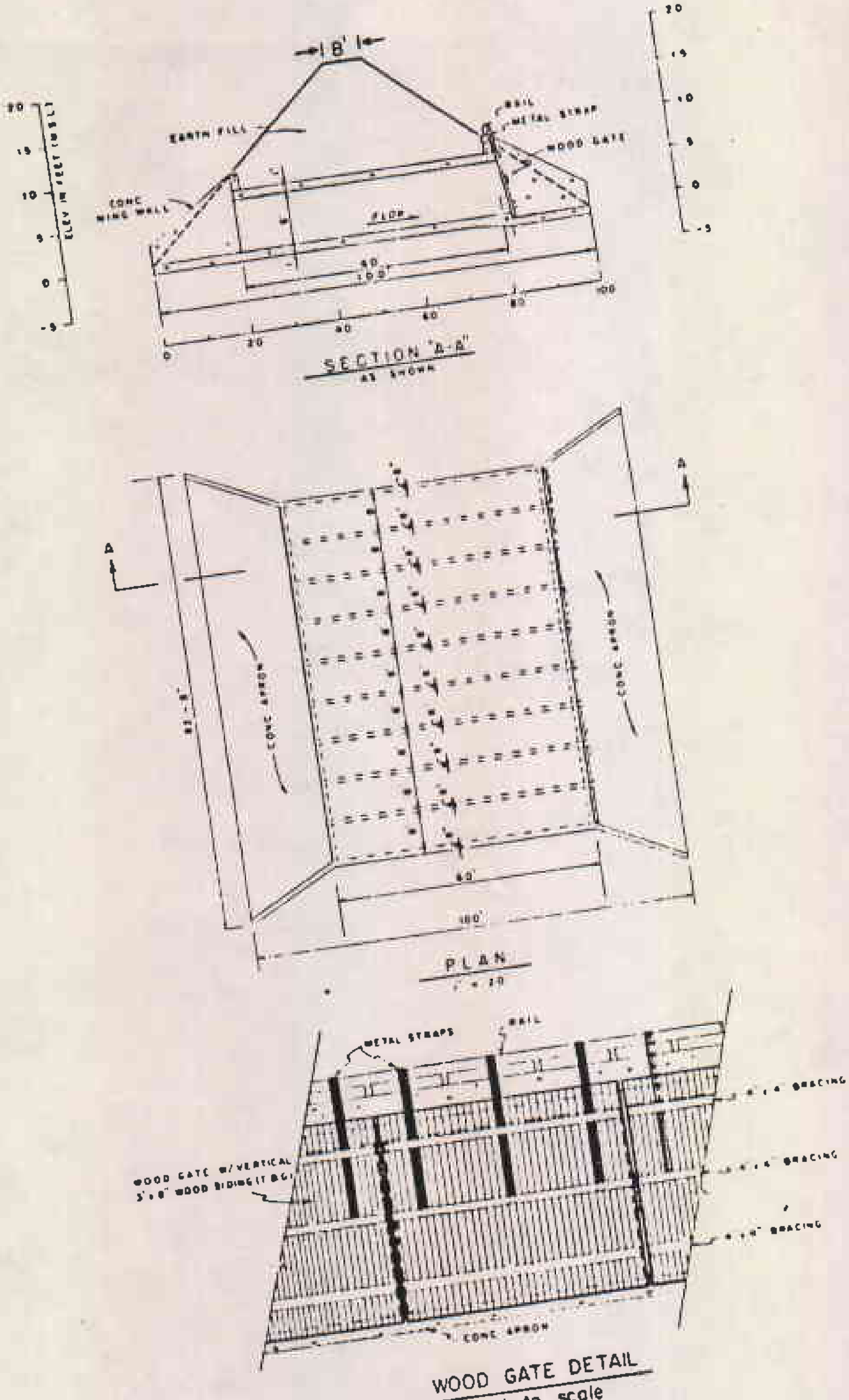


Figure 35
TRANSVERSE DIKE WITH TIDE GATES

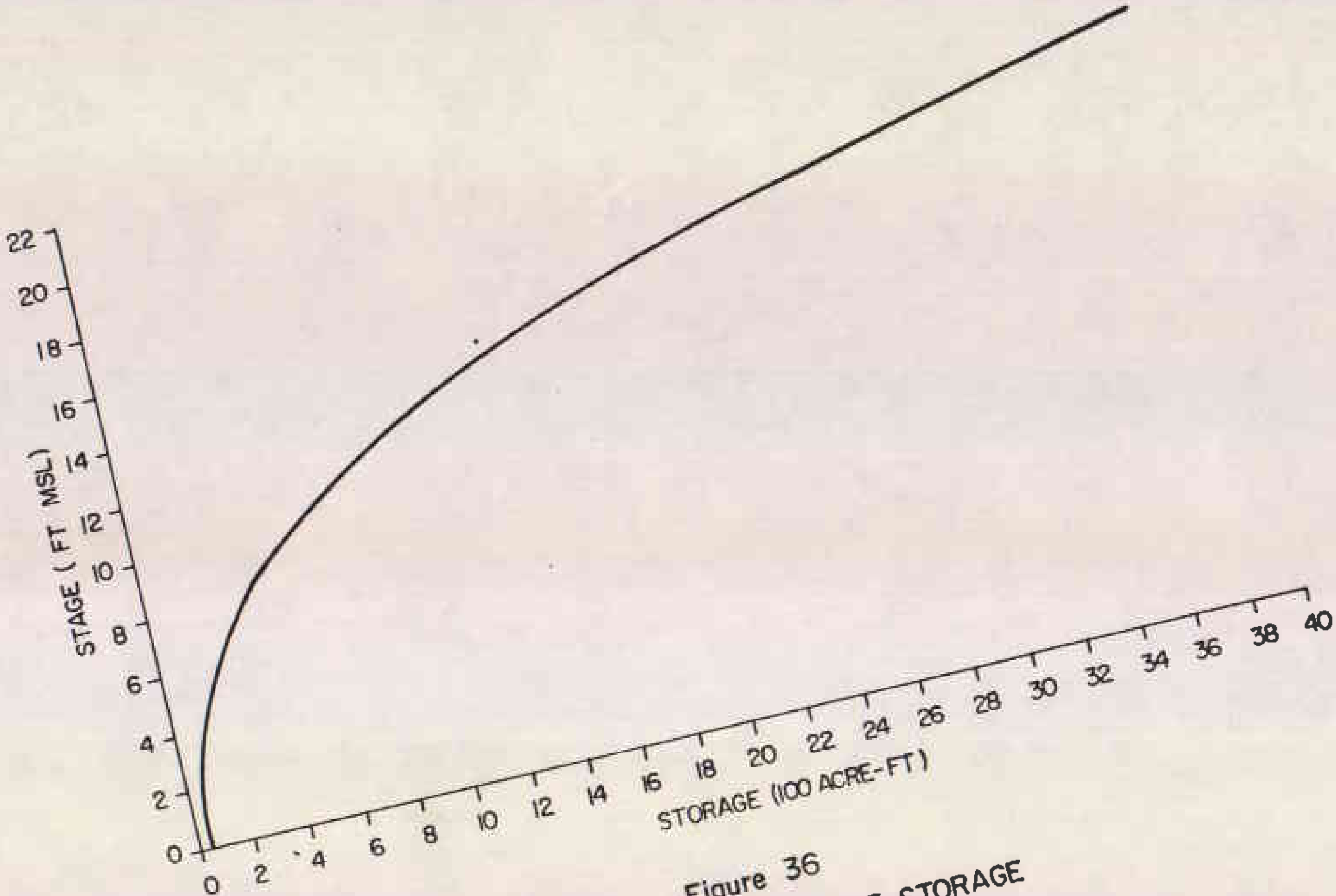


Figure 36

TURKEY CREEK BASINWIDE STAGE STORAGE

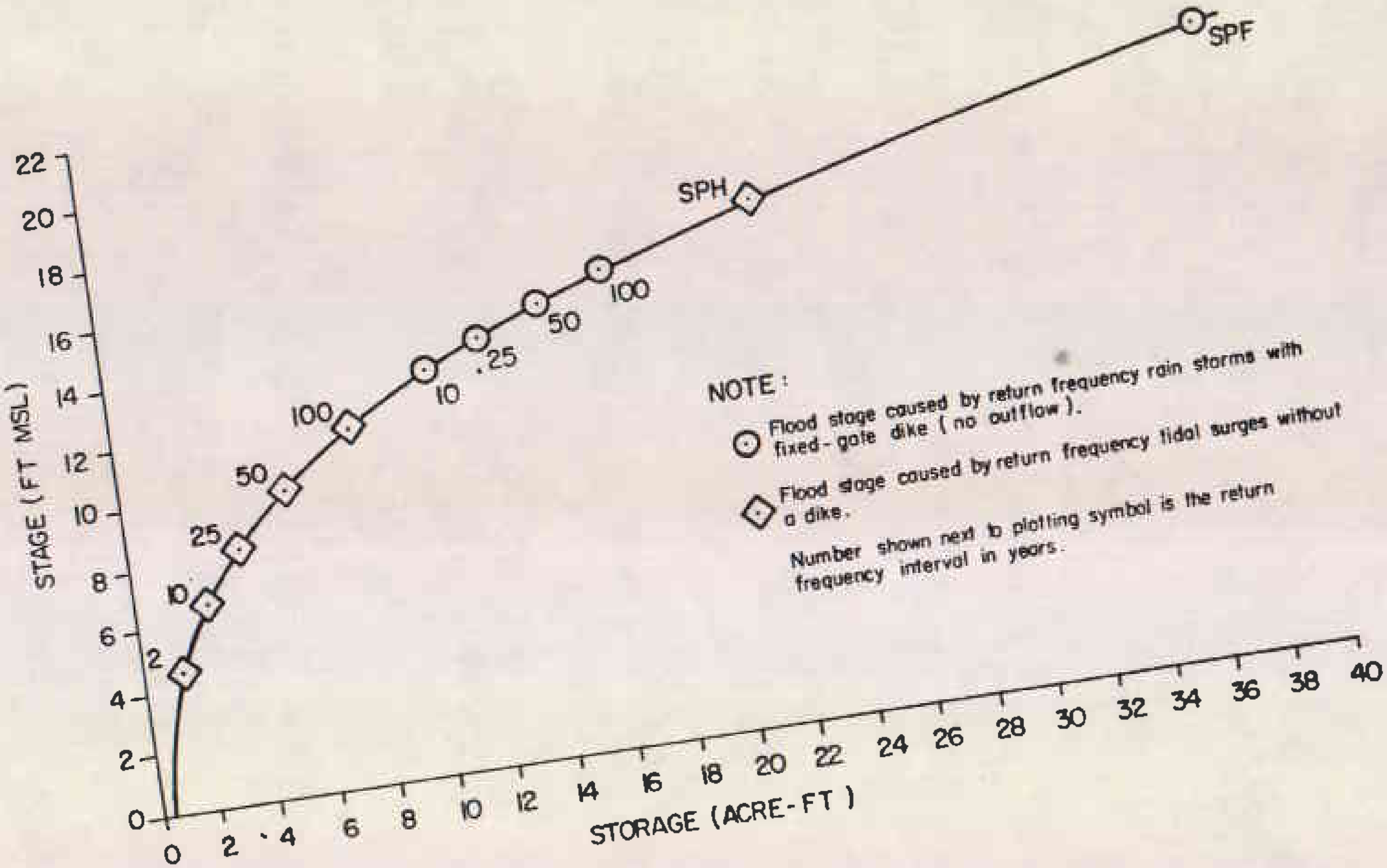


Figure 37

TURKEY CREEK FIXED-GATE DIKE FLOOD STAGES

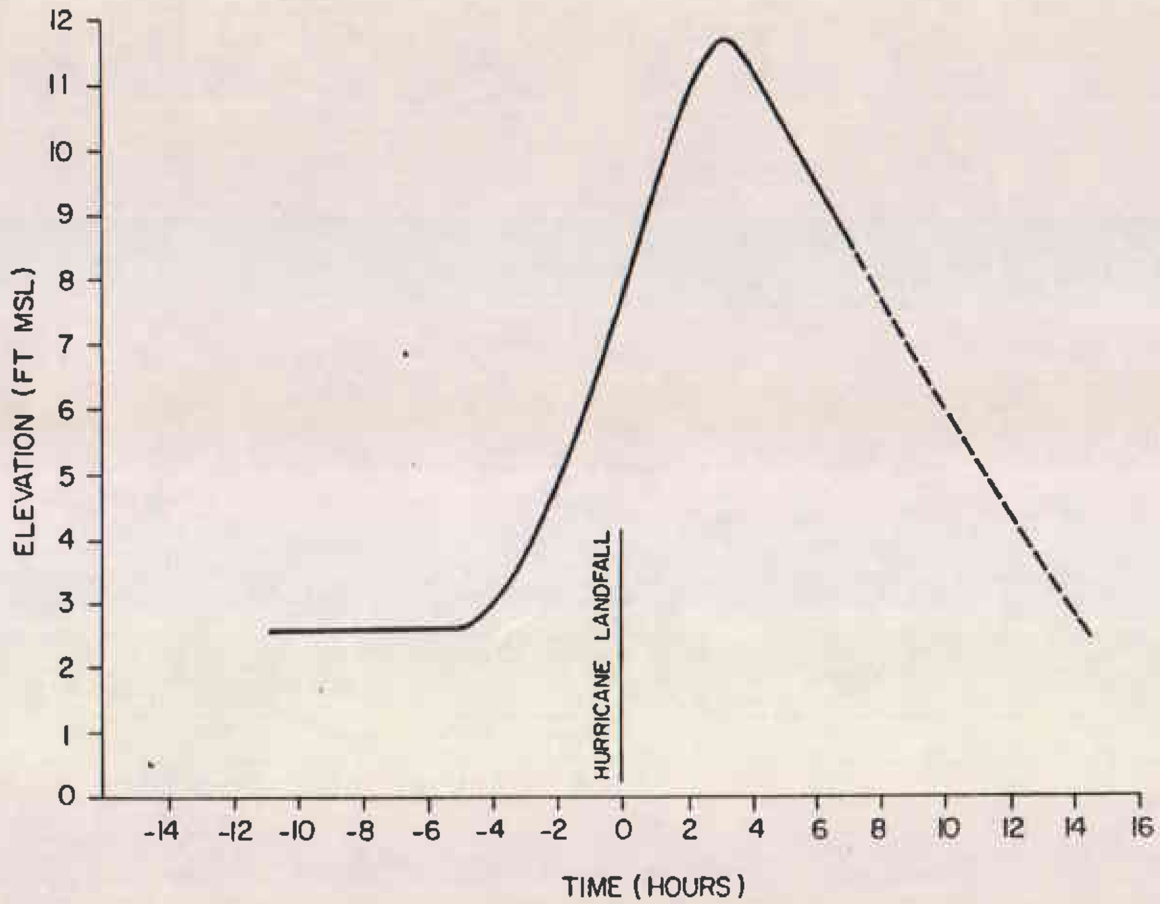


Figure 38

100 YEAR HURRICANE TIDAL STAGE AT MOUTH
OF GOOSE CREEK