

Duke Power Report 1940 Flood

Contents:

File Name: 88_1940_Report_1

Note:

DUKE POWER COMPANY

P. O. BOX 2178

GENERAL OFFICES
422 SOUTH CHURCH STREET
CHARLOTTE, N. C. 28201

TELEPHONE: AREA 704
332-8521

April 18, 1969

Mr William H Hulbert
Hydrologist
South Carolina State Highway Department
Drawer 191
Columbia South Carolina 29202

Dear Mr Hulbert:

Please refer to your March 27 letter requesting information on Catawba River floods. We understand you are trying to set critical clearances at I-77 Bridge crossing near Rock Hill S C.

Major floods have occurred on the Catawba in 1908, 1916, 1929 and 1940. The 1916 or 1929 flood is the one of record below Lookout Shoals dam in North Carolina and the 1940 flood is the one of record above this dam. During the 1916 flood, Duke had completed two dams above Rock Hill, one being the Old Catawba which has since been rebuilt and raised and the other being Lookout Shoals dam. A washout occurred at Lookout Shoals dam during the 1916 flood. Additional information concerning the 1916 flood may be found in a book published in 1917 by the Southern Railway Company titled "The Floods of July 1916." During the 1940 flood, Duke had six dams constructed above Rock Hill which tended to smooth out flood flows and minimize damage. Since 1940, Cowans Ford Station has been completed.

We are enclosing the following information which may be of help to you:

- 1) Profile of Catawba River showing 11 impoundments along 220 miles of the river.
- 2) Tabulation of Flood Flows and Elevations corresponding to the 1916 and 1940 floods at the Duke hydroelectric stations.
- 3) Tabulation showing Plan View of Drainage Basin and Reservoir and Drainage Areas.
- 4) Profiles of Fishing Creek and Wylie Stations with 1916 floodwater line and deduced water surfaces assuming these stations had been constructed prior to 1916 flood.

Please advise if you have further questions or require additional information.

Yours very truly,



L C Dail, Principal Civil Engineer

LCD-i
Encl

For Rt 21
Rocky Cr.

Datum of Plans ↘

BM 39 Proj 370-D = Elev 474.73
Proj 399-(8) = Elev 482.01 = USGS MSL.
7.28

Spillway = 289.4 MSL = 277.12 schD

1990 Hw = 290.2 = 282.92

From Ben Whetstone USGS.

Cedar Cr Reservoir

DA. 4360 sq. mi.

Plans Datum

Mean Flow 5420 cfs

Spillway design flow 343,300 cfs

Elev top of dam 306.4

299.12

top of Gate 286.1

278.82

crest of spillway 284.9

277.12

max power pool elev 284.9

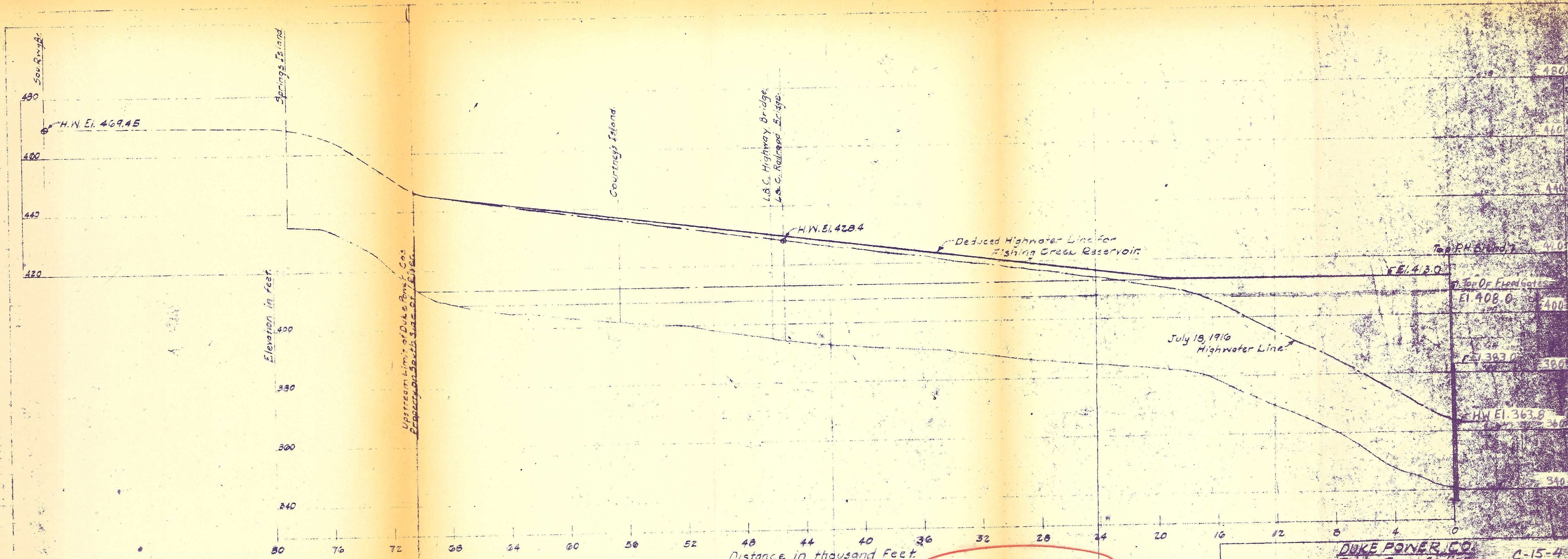
277.12

min power pool elev 282.9

275.12

max water elev 290.2

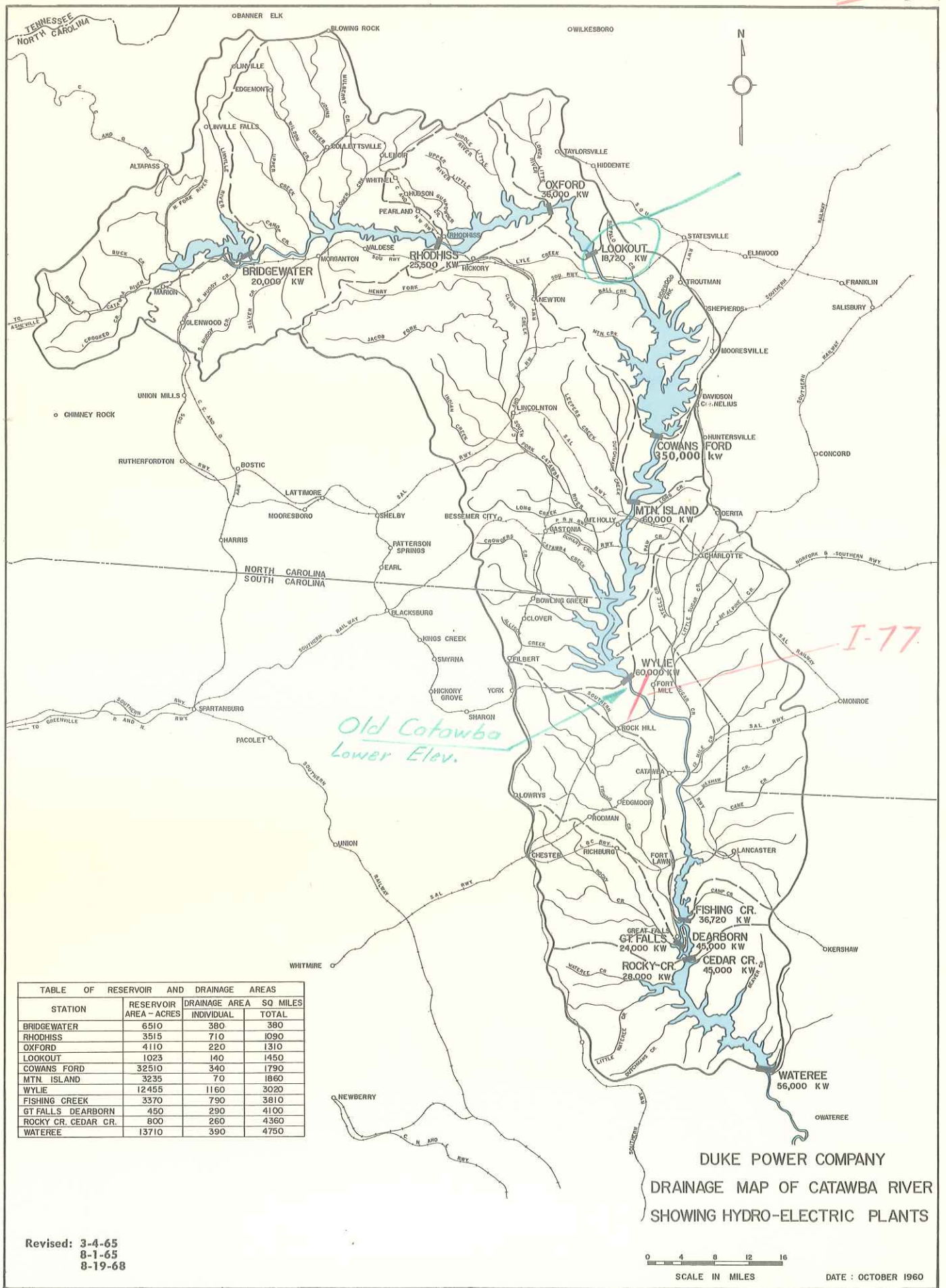
282.92



Note: To refer elevations shown on profiles
 - U.S.G.S. Elevations add 0.8 feet.
 ⊕ Denotes actual water level determined in field.
 The term "River Bed" on Profile refers to Low Waterline in River Reach considered.

TO CONVERT ELEVATIONS TO U.S.G.S. DATUM
 ADD 0.8

DUKE POWER CO. C-15-4
 Fishing Creek Station
 Profiles Showing July 18, 1916 Highwater Line of Corawba and Deduced Highwater Line for this same Flood Discharge Considering Dam at Fishing Creek Station
 PFC-39



Revised: 3-4-65
8-1-65
8-19-68

0 4 8 12 16
SCALE IN MILES

DATE : OCTOBER 1960

DUKE POWER COMPANY
DRAINAGE MAP OF CATAWBA RIVER
SHOWING HYDRO-ELECTRIC PLANTS

FLOOD FLOWS AND ELEVATIONS AT DUKE HYDRO PLANTS

CATAWBA RIVER	Drain Area, Sq Mi	Flood of July 1916				Flood of Aug 1940				
		Cfs Total	Cfs/Sq Mi	Water Elevn	Dam Crest	Cfs Total	Cfs/Sq Mi	Water Elevn	Dam Crest	
Bridgewater	380	75,200	198	1111	*	141,800	373	1206.1	1200.0	Brdwatr
Rhodhiss	1090	149,100	137	968	*	161,600	148	1004.9	995.1	Rhodhis
Oxford	1310	165,000	126	881	*	189,800	145	939.5	935.0	Oxford
Lookout	1450	181,000	125	860	838.1	211,500	146	852.5	838.1	Lookout
Catawba N C	1535	199,500	130	?	None	177,000	115	?	None	Cat N C
**Cowans Ford	1790	-	-	-	*	-	-	-	760.0	Cow Frd
Mtn Island	1860	200,000	108	614	*	121,700	65	657.0	647.5	Mtn Isl
Wylie (Catawba)	3020	299,400	99	548	520.5	166,300	55	569.4	569.4	Wylie
Fishing Creek	3810	341,900	90	364	*	113,900	30	416.8	417.2	Fsh Crk
Gt Falls-Dearbn	4100	363,300	89	?	355.8	?	?	?	355.8	GF-Dbn
Rky Crk-Cdr-Crk	4360	383,500	88	?	284.4	114,200	26	290.2	284.4	RC-CC
Wateree	4750	415,000	87	177	*	133,800	28	231.8	225.5	Wateree
<u>BROAD RIVER</u>										
Gaston Shoals	1250	?	?	614.5	?	109,000	87	613.3	605.4	Gaston
99 Islands	1550	?	?	524.6	?	121,400	78	523.1	511.1	99 Islld

? = unknown

* Since these dams were not built until after 1916, the water-surface elevations are those of the unobstructed river. As a partial exception to this statement, the Fishing Creek spillway was under construction at the time of the 1916 flood. The two spillways whose construction was nearest in time to this flood were at Bridgewater, started in Aug 1916, and at Wateree, begun in Feb 1917.

** There are no flood records at Cowans Ford site. The 1916 and 1940 flood-flow records nearest there are those at the USGS gaging station at Catawba N C, about 21 river-miles upstream, and they are approximations.

All dams except Cowans Ford were built before the 1940 flood.

In July 1916, the crest of the spillway at original Catawba was at elevation 520.5, but was raised in 1924-5 to 569.4. In 1960, the plant name was changed to Wylie.

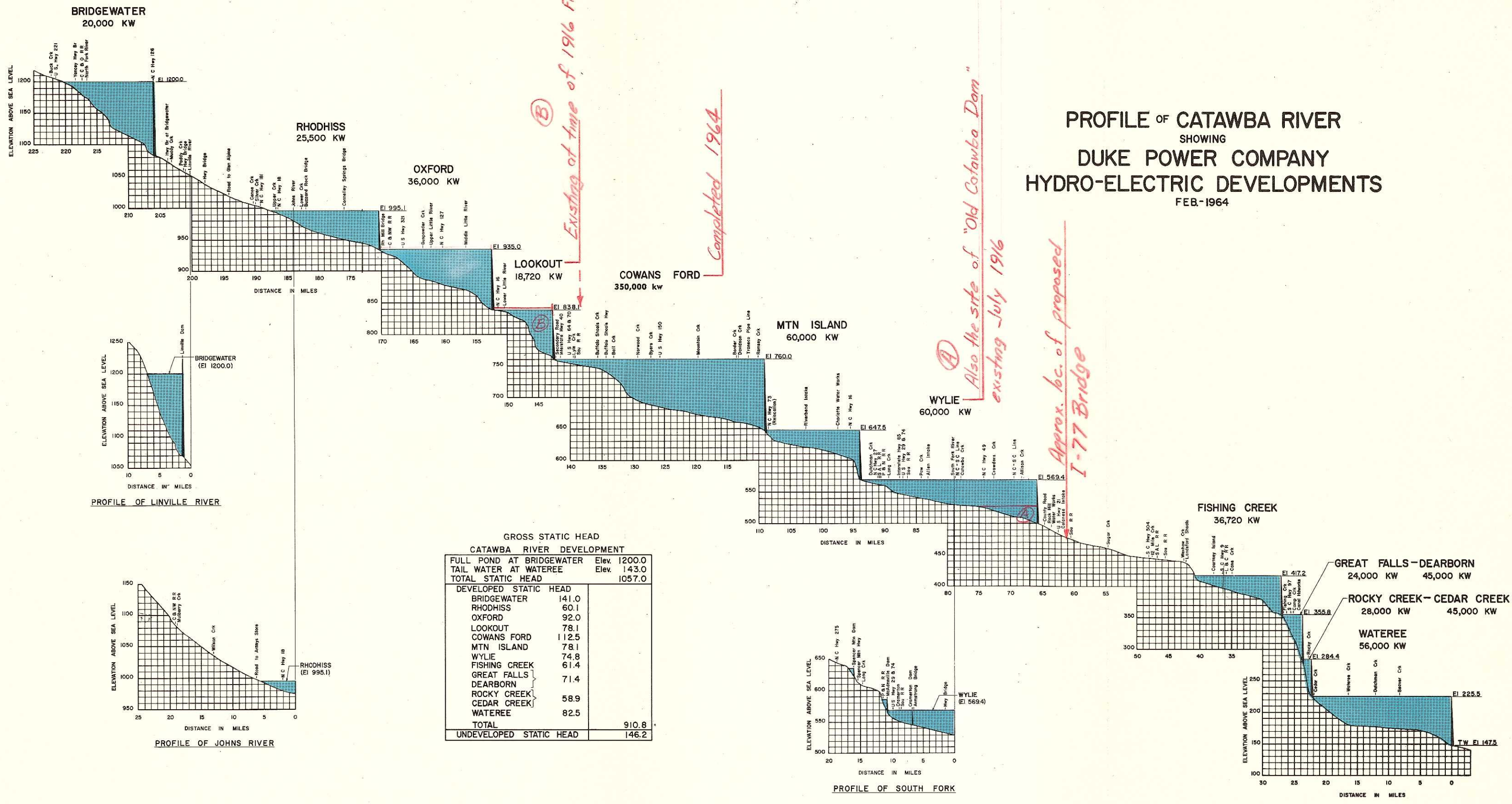
At Gaston and Ninety Nine, the 1940 dam-crest elevations are the top of flashboards, the heights being 6.0 and 2.0 feet respectively. The 1916 flashboard heights are unknown, but the concrete dam crests were then, as now, at 599.4 and 509.1

The drainage areas here shown agree with those of the Drainage Map on Manual page D-1.

PCG 4-21-61. Added flood elevns & notes 11-10-61, added 1940 data for Gaston & 99 Isl in File S-22
1962, added GS & 99 Isl crest elev & 1940 cfs/sq mi 2-19-66.
Revised 2-4-65, 2-21-66; ok 2-08-67

PROFILE OF CATAWBA RIVER

SHOWING
DUKE POWER COMPANY
HYDRO-ELECTRIC DEVELOPMENTS
FEB-1964

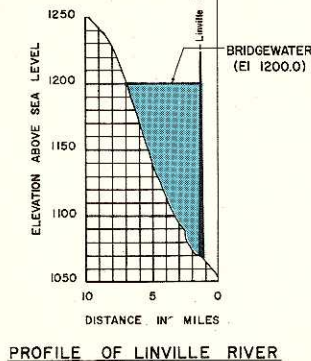


(B) Existing of time of 1916 Flood

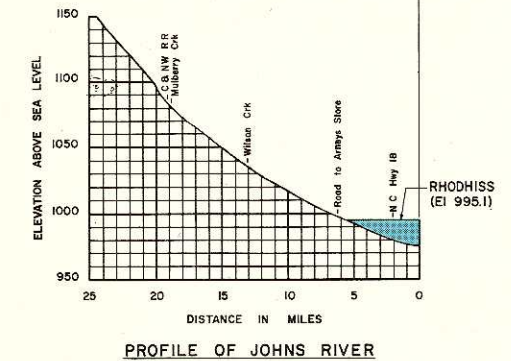
Completed 1964

(A) Also the site of "Old Catawba Dam" existing July 1916

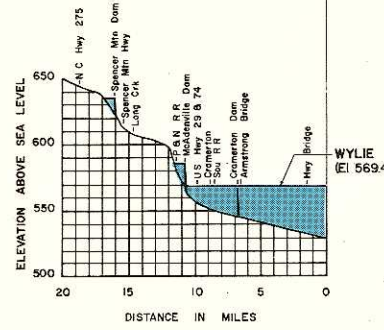
Approx. loc. of proposed I-77 Bridge



PROFILE OF LINVILLE RIVER



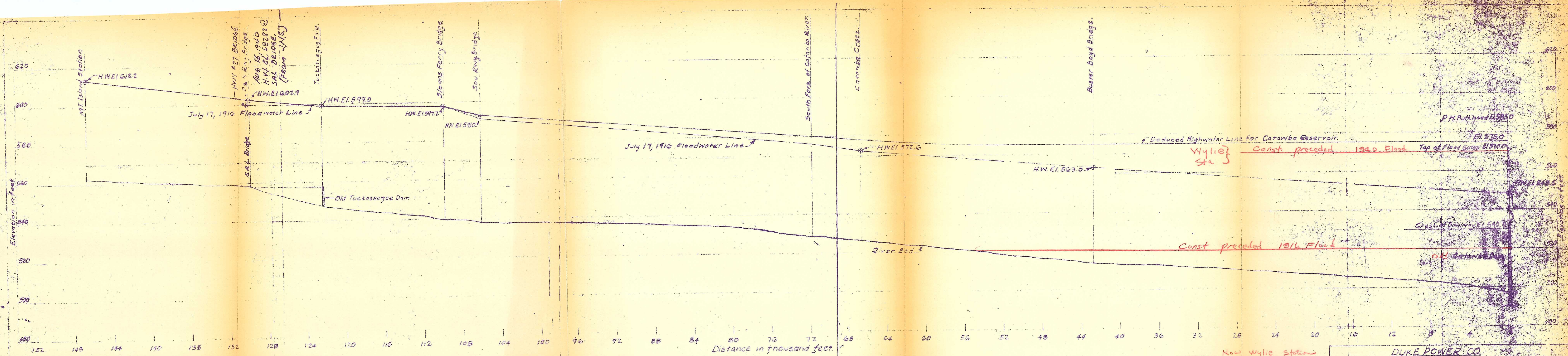
PROFILE OF JOHNS RIVER



PROFILE OF SOUTH FORK

GROSS STATIC HEAD	
CATAWBA RIVER DEVELOPMENT	
FULL POND AT BRIDGEWATER	Elev. 1200.0
TAIL WATER AT WATEREE	Elev. 143.0
TOTAL STATIC HEAD	1057.0
DEVELOPED STATIC HEAD	
BRIDGEWATER	141.0
RHODHISS	60.1
OXFORD	92.0
LOOKOUT	78.1
COWANS FORD	112.5
MTN ISLAND	78.1
WYLIE	74.8
FISHING CREEK	61.4
GREAT FALLS	71.4
DEARBORN	71.4
ROCKY CREEK	58.9
CEDAR CREEK	58.9
WATEREE	82.5
TOTAL	910.8
UNDEVELOPED STATIC HEAD	146.2

MK'd 4-18-69



Note: ⊕ Denotes actual water elevation determined in field. The term 'River bed on Profile refers to Low Waterline in River Reach considered.

Now Wylie Station
 TO CONVERT ELEVATIONS TO U.S.S. DATUM SUBTRACT 0.8 FT.

DUKE POWER CO.
New Catawba Station, Wylie Station
 Profiles showing July 17, 1916 Highwater Line of Catawba River and Deduced Highwater Line for 1940 Flood Discharge Considering Dam at New Catawba Station.
 PNC41 6-13-33

USGS Report 1940 Flood

Contents:

File Name: 88_1940_Report_2

Note:

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Co.

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SAVANNAH RIVER NEAR MILLHAVEN, GEORGIA
Flood-Flow Characteristics at Bridge Site

U. S. Route 301

Screven County

FOR ADMINISTRATIVE USE ONLY

Atlanta District
Water Resources Division
U. S. Geological Survey
Atlanta, Georgia

January 1961

Prepared by: C. M. Bunch Hydraulic Engineer

M. Price Hydraulic Engineer

Approved by: H. H. Odell Acting Dist. Engineer

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SAVANNAH RIVER NEAR MILLHAVEN, GEORGIA
Flood-Flow Characteristics at Bridge Site
U. S. Route 301
Screven County

INTRODUCTION

The State Highway Department of Georgia proposes, as a joint undertaking with the South Carolina Highway Department, the construction of a new crossing of the Savannah River on U. S. Route 301, at Burtons Ferry near Millhaven, Georgia. The Georgia Highway Department has requested an analysis of flood-flow characteristics at the proposed crossing site. This report has been prepared by the Atlanta District, Water Resources Division, of the U. S. Geological Survey, in compliance with the request and under authority provided in a cooperative agreement between the two agencies.

The proposed construction will be the second lane of a dual lane crossing. The existing bridges and embankments will, with some proposed modification, form the other lane. The proposed work will generally be a short distance upstream and parallel to the existing crossing. The general location of the proposed crossing is shown in the sketch of figure 1. The aerial photograph of plate 1 shows the valley reach in the immediate crossing vicinity and features of channel and topography which may be pertinent to waterway opening design.

The Savannah River at this point drains an area of 8,650 square miles. Flood discharge is controlled to an appreciable extent by Clark Hill Reservoir, which was put into operation in December 1951, and will be further controlled by Hartwell Reservoir on which construction is now in progress. Clark Hill Dam is about 95 miles upstream where the drainage area is 6,140 square miles and Hartwell Dam is about 160 miles upstream where the stream drains an area of 2,090 square miles.

The South Carolina Highway Department has indicated a specific interest in the peak discharge for the October 1929 flood as it would be modified by operation of Clark Hill and Hartwell Reservoirs had they been in operation. They provided aerial photographs of the valley reach, a profile of the valley along the existing crossing center line, and plans showing existing and proposed abutment locations.

This report presents an analysis of the hydraulic features of the channel and floodway and relates these features to flood discharge and other hydrologic information that may be pertinent to the design of the waterway openings. The hydrologic information for natural or unregulated conditions is based on data summarized in Geological Survey Circular 100, Floods in Georgia (1950). Information relative to effects of reservoir operation on the magnitude of flood discharge has been provided by the Savannah District Office of the Corps of Engineers. Hydraulic computations have been based on channel and floodway conditions as observed in December 1960.

All elevations mentioned refer to highway department datum unless specifically stated otherwise. To convert highway datum to mean sea level, datum of 1929, subtract 0.6 foot.

The data, computations and correspondence supporting the information given in the text, tables and illustrations are available for inspection at the U. S. Geological Survey District office in Atlanta, Georgia.

MAGNITUDE AND FREQUENCY OF FLOODS

Flood Discharge

The Corps of Engineers has operated a gage at Burtons Ferry Bridge, near Millhaven, Georgia since October 1939. Records of discharge are published in the Water Supply Papers of the Geological Survey. Records for a longer period are available for upstream and downstream stations at Augusta, Georgia and Clyo, Georgia.

The flood of September-October 1929 produced the highest stages known on the Savannah River. The October 1929 crest at Augusta is known to be the highest since 1796. Notable floods in recent times occurred in 1936 and 1940.

According to the Corps of Engineers the flood of October 1929 produced a peak discharge of 220,000 cubic feet per second at a crest elevation of 83.8 feet at Burtons Ferry Bridge. There is hearsay evidence that the 1936 flood crested at 81.6 feet. The peak discharge for the 1940 flood was 141,000 cfs at a crest elevation of 80.0 feet.

The maximum peak discharge at this site since Clark Hill Reservoir was put into operation in 1951 was 41,000 cfs in April 1958.

Flood Frequency

Frequency of flood discharge on the natural unregulated river at Burtons Ferry Bridge is defined by the annual floods for the period 1940-49 (adjusted to the period 1892-1949) and discharge-frequency relationships as presented in Circular 100, Floods in Georgia. The estimated recurrence intervals of discharges for natural, unregulated conditions are shown by the flood-frequency curve of figure 2. This curve is based on data listed and computation procedures outlined in Circular 100 and indicates a discharge of an expected 50-year frequency of discharge to be 212,000 cfs.

Effect of Reservoir Operation

Since Clark Hill and Hartwell Reservoirs would often be drawn down when floods occur, the normal operation of the reservoirs would generally result in a reduction of flood peak discharges downstream in addition to reduction due to storage allotted to flood control.

The following is quoted from a written communication from the Savannah District of the Corps of Engineers, dated 4 November 1960:

"With both Hartwell and Clark Hill Projects in operation, it is estimated that the peak discharge of the 1929 flood would be 120,000 cfs, which corresponds to a stage of 25.8 [78.8 ft.], and the peak discharge for the 1936 flood would be 80,000 cfs,

corresponding to a stage of 22.8 feet. [75.8 ft.]-----
The actual peak stage of the 1929 flood was 30.8 [83.8 ft.]
(220,000 cfs) while the peak of the 1936 flood at Burton's
Ferry is unknown."

It is assumed the estimate of reduction in specific flood
peaks was arrived at by routing the 1929 and 1936 floods through
the reservoirs and down river to Burtons Ferry Bridge.

Accompanying the above quoted communication was a flood-
frequency curve indicating the Corps' estimate of discharge-frequency
relationship at Burtons Ferry Bridge with both reservoirs in operation.
This curve indicates discharges of 50-year and 100-year expected
frequency of recurrence to be 67,000 cfs and 80,000 cfs respectively.
The methods used to define this curve are not known.

STAGE DISCHARGE RELATION

During the period of gaging-station operation the relation
of stage to discharge has been defined by current-meter measurements
up to 140,000 cfs. The curve of figure 3, showing this relationship,
has been extended to 220,000 cfs on the basis of the information pro-
vided by the Corps of Engineers.

Indicated on figure 3 are elevations at which October 1929,
100-year and 50-year discharges, as estimated by the Corps of Engineers,
would occur.

DISTRIBUTION OF DISCHARGE

A profile of the natural valley along the centerline of the existing crossing is shown in figure 4A. Figure 4A also shows abutment locations of existing bridges. The plan of the existing crossing is shown in figure 4B, with degree and limits of curvature indicated. The proposed crossing site was visited and studied in December 1960. There was no evidence of scour at any of the bridge openings and there was no evidence of any significant change having been made in the valley cover in recent years, such as might be caused by logging operations.

The Corps of Engineers has obtained some high-stage current-meter measurements of discharge through the existing openings during the period of gaging station operation. It is thought that an analysis of these discharge measurements would give the most reliable estimate of distribution of discharge through the proposed openings. The following listed discharge measurements are available for study:

<u>Meas. No.</u>	<u>Date</u>	<u>Stage</u>	<u>Elevation</u>	<u>Discharge</u>
38	8-18-40	27.0	80.0	140,500
80	1-13-46	21.3	74.3	65,000
90	1-26-47	21.4	74.4	64,000

Significant data taken directly from measurement 38 are shown in figure 4B.

From information contained in the discharge measurements, stage-discharge, stage-area and stage-velocity curves were drawn

for each opening, and for the main-channel and overbank sections of the main bridge opening. The data shown in figure 4C were interpolated from these curves to give the discharge distribution and average velocities in the existing openings for the design discharge of 120,000 cfs.

The distribution as actually measured for a total discharge of 140,500 cfs, as reflected by measurement 38 and shown in figure 4B, was checked against distribution as computed by a procedure that would be used where measured distribution is not available. In this procedure it is assumed that the centerline section is representative of the approach reach and the total discharge is distributed over the valley in proportion to conveyance distribution in the centerline section. The discharge is then distributed among the several openings on the basis of distribution in the approach reach and the size and geometry of the bridges. The variation of computed discharge from measured discharge did not vary more than three percent for any of the existing openings. It would seem then that the computational procedure should give a fairly reliable estimate of distribution through openings as proposed for the new construction.

As previously stated, the new construction will be generally parallel to and upstream from the existing crossing. Proposed bridges will be 70 feet upstream, centerline to centerline, from existing bridges with the exception of the main channel bridge which will be about 140 feet upstream. Abutments will be of the open end bent,

spill through type with two on one slopes. The existing bridges will be shortened so that abutment stationing will correspond to that of the proposed openings and abutment slopes will be continuous through new and existing bridges.

Figure 5A shows the valley profile with proposed waterway opening locations. Plan of proposed work is shown in figure 5C with modified curvature noted.

Figure 5B shows the computed distribution of the regulated 1929 peak discharge (120,000 cfs) across the valley. This computation is based on the given profile and channel and floodway conditions as observed in December 1940. That portion of the total flow allotted to the channel on the far right edge of the valley (stations 153+00 to 158+00) is an approximation based on discharge measurement results. This will be discussed further in a later section of this report.

Figure 5C shows the computed discharge distribution among the several openings as proposed. This computation is by the procedure described earlier in this section.

COMMENTS-GENERAL

The high ground indicated at sta. 47+00 in figures 4A and 5A does not extend either upstream or downstream from the existing crossing at this time. The type of cover on a rather large area immediately downstream indicates that there may have been a ridge extending downstream at the time the crossing was constructed and

the material was probably used in the existing embankment.

The ridge indicated at sta. 85+00 apparently extends far enough upstream to be effective as a flow divide between adjacent overflow bridges for crests under 79.0 ft.

The channel on the extreme left edge of the valley is Brier Creek (South Carolina) and, at the crossing site, is separated from the Savannah River floodways by the ridge extending from sta. 151+50 to sta. 153+50. Available maps are not in sufficient detail to show the extent of this ridge upstream and downstream.

As mentioned on the preceding page, the 500 cfs shown as the discharge through the Brier Creek opening (fig. 5B-C) when the total discharge is 120, 000 cfs is an approximation based on the measured discharge of 1,000 cfs when measurement no. 38 was made. Because of a berm ditch along the upstream embankment toe, lateral flow is possible and it is highly probable that a large portion of the measured flow of 1,000 cfs through this opening was Savannah River water rather than flow from the Brier Creek Basin. This is more likely when it is considered that flood peaks on Brier Creek (drainage area about 24 square miles) would generally occur hours or even days before a crest on the main river.

After construction as proposed, the existing upstream berm ditch will be covered by the proposed roadway and the ridge at station 151+50 to 153+50 would apparently prevent lateral movement of flow as is now possible.

In view of the above, in designing a structure at this location, maybe the design should be based on expected flow from Brier Creek Basin. It is possible that the Geological Survey office in Columbia, South Carolina may be able to provide information relative to magnitude and frequency of floods in drainage areas of this size. Unless some observations of stage and discharge have been made here, it would probably be necessary to make a survey of channel properties in order to establish an approximate stage-discharge relationship.

It is noted that the South Carolina Highway Department has tentative plans to construct a triple 10 ft. by 8 ft. box culvert for the Brier Creek opening. The proposed elevation of the top of the barrel is 83.0 feet. It would appear highly improbable that headwater elevation would ever exceed by any significant amount the 81.0 foot elevation of the ridge separating the Brier Creek channel from the Savannah River floodway. In other words, the upper two feet of proposed culvert barrel would probably never be effective.

SUMMARY

1. The peak discharge of the October 1929 flood was 220,000 cfs at a crest elevation of 83.8 feet.
2. Corps of Engineers studies indicate that, had Clark Hill and Hartwell Reservoirs been in operation, the peak discharge of the October 1929 flood would have been 120,000 cfs which corresponds to an elevation of 78.8 feet.

3. The natural, unregulated discharge of an expected 50-year frequency of recurrence is estimated as 212,000 cfs.

4. The Corps of Engineers estimates that, with both reservoirs in operation, peak discharges of 50-year and 100-year expected frequency of recurrence are 67,000 cfs and 80,000 cfs respectively.

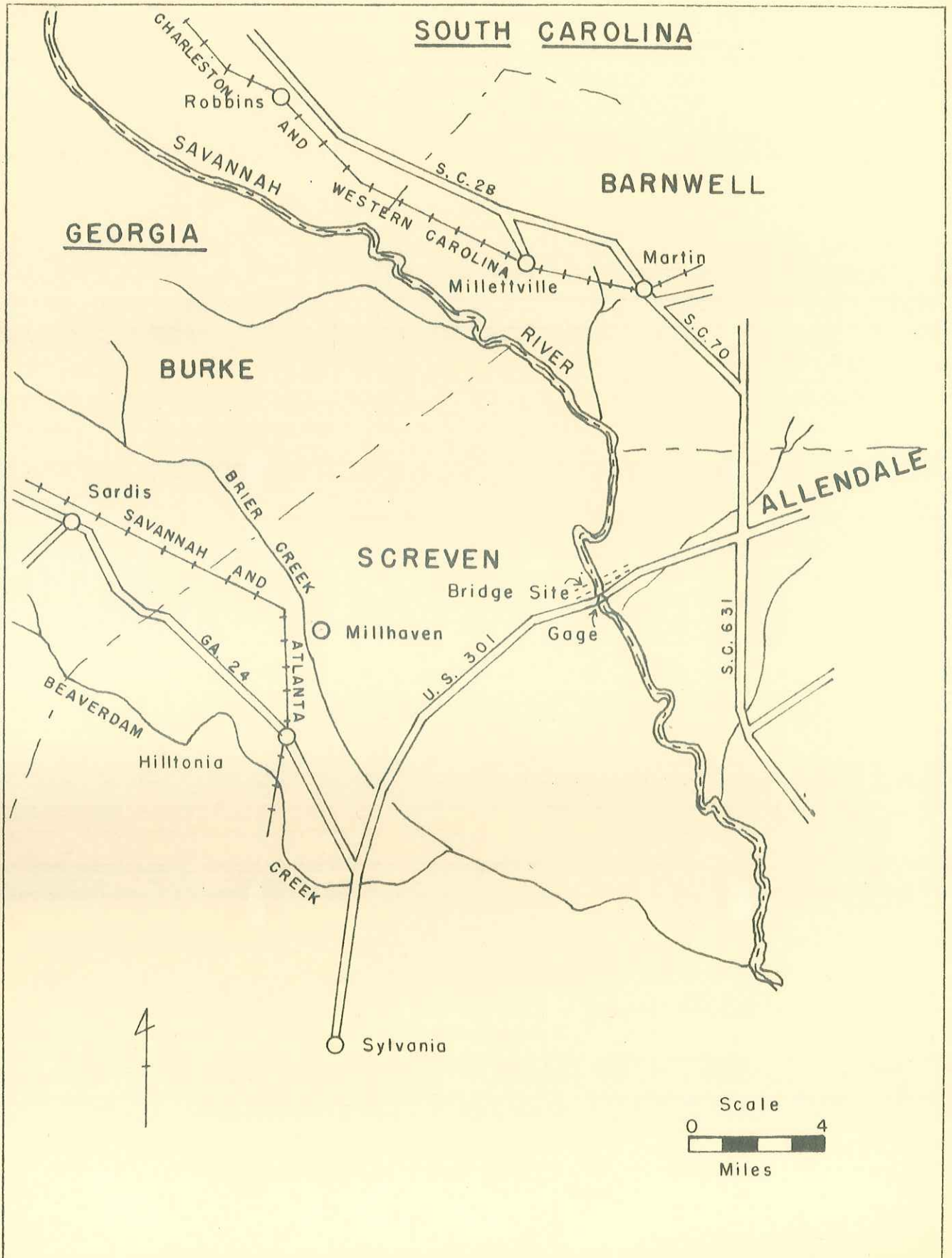
5. Computed distribution of a 120,000 cfs discharge through openings as proposed is shown in figure 5C of this report.

6. Some consideration might well be given to treating Brier Creek flood flow as a separate problem in the crossing design.

Plate I Aerial View Savannah River - Route 301



Figure 1 Location Sketch Savannah River - U.S. Route 301



Sheet No. _____ of _____ Sheets. Prepared by M.P. Date 1-61 Checked by C.M.B. Date 1-61

Figure 2 Flood-frequency Curve

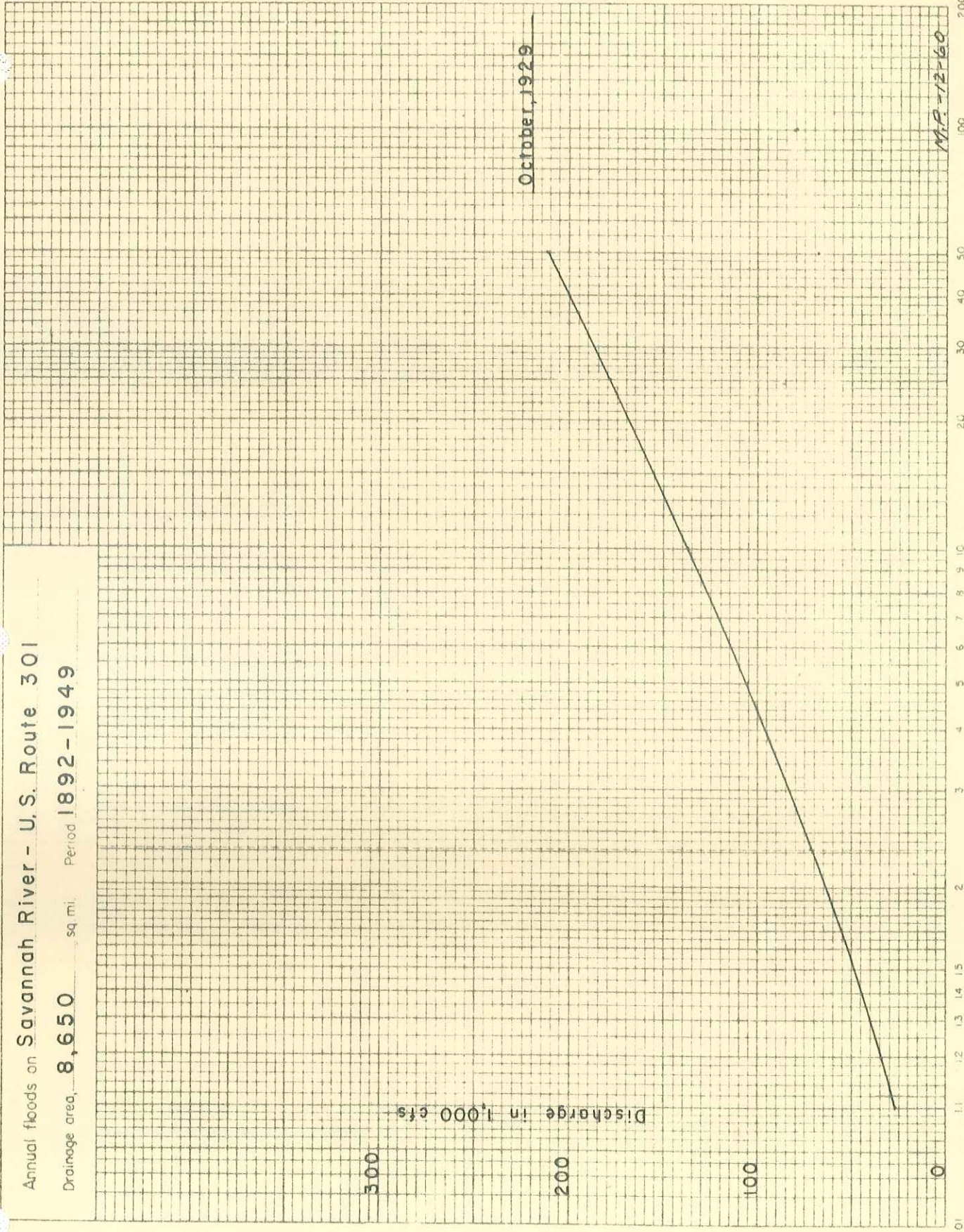
File Number

UNITED STATES DEPARTMENT OF THE INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

9-179a
Flood data plot
(March 1949)

Annual floods on Savannah River - U.S. Route 301

Drainage area, **8,650** sq. mi. Period **1892-1949**



Recurrence interval, in years

Figure 3 Stage-Discharge Curve Savannah River - U.S. Route 301

(U.S.E.D.)

16-13408-2 GPO

(DO NOT USE THIS SPACE EXCEPT FOR BINDING PURPOSES)

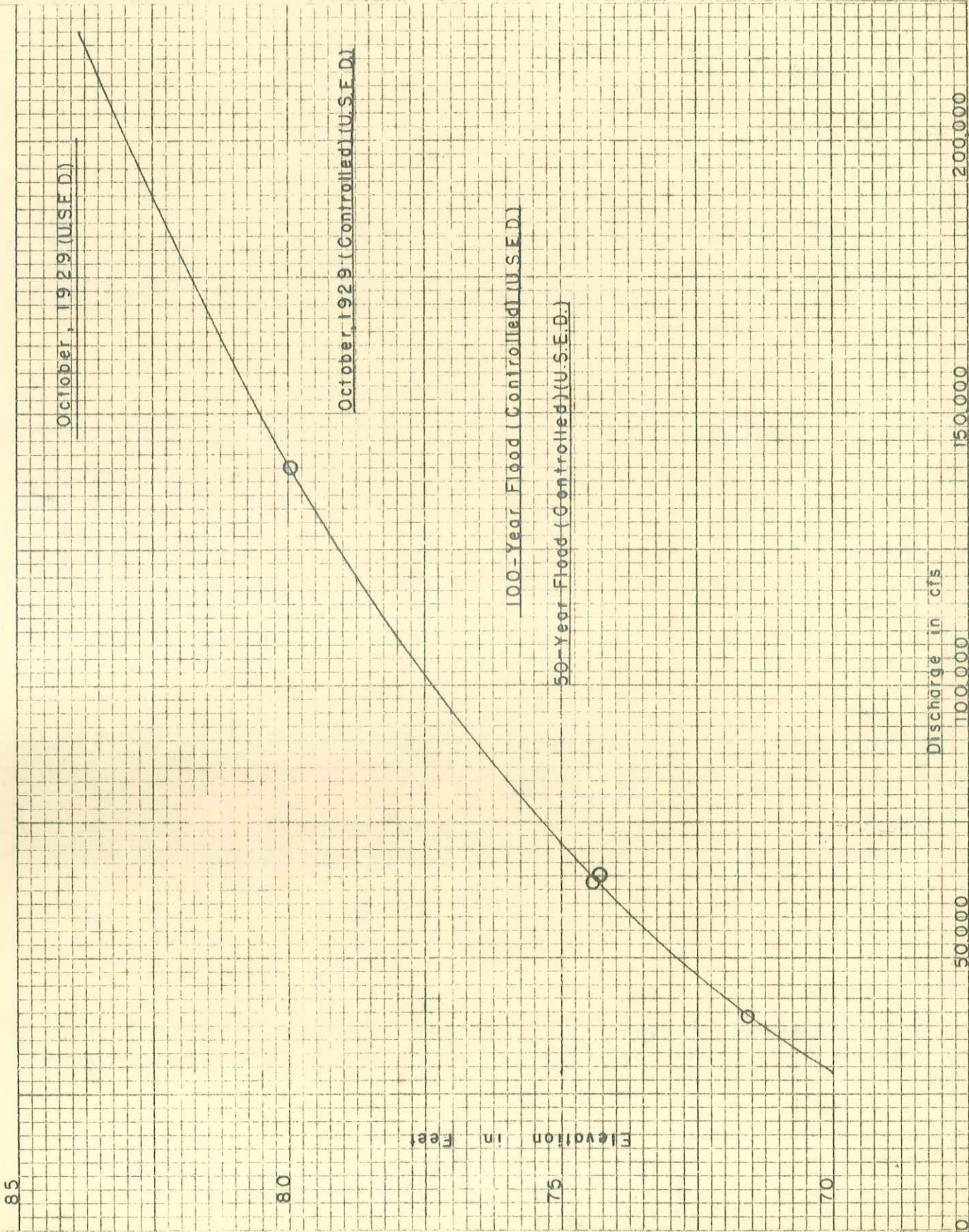


Figure 4 Analyses of Flow Distribution through existing bridges of Savannah River at Route 301

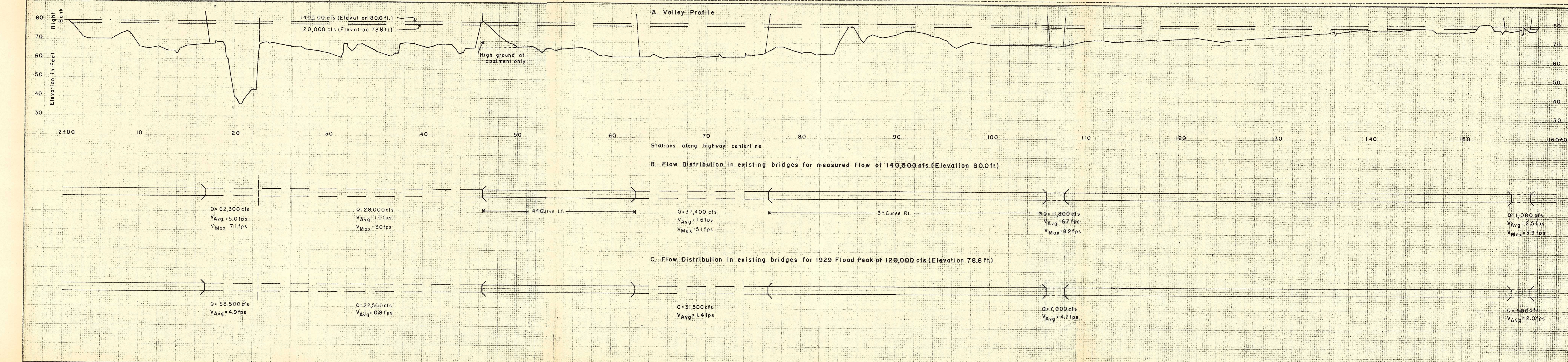


Figure 5 Analysis of Flow Distribution through proposed bridges of Savannah River at Route 301

