INVESTIGATION OF SELECTED STREAMFLOW CHARACTERISTICS OF THE ALABAMA RIVER UPSTREAM FROM SELMA, ALABAMA

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 85-4055

Prepared in cooperation with the

U.S. ARMY CORPS OF ENGINEERS, MOBILE DISTRICT

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Montgomery, Alabama

1985

UNITED STATES DEPARTMENT OF THE INTERIOR

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GEOLOGICAL SURVEY

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CONVERSION FACTORS

For use of readers who prefer to use metric units, conversion factors for terms used in this report are listed below:

Multiply	By	To obtain
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
foot per second (ft/s)	0.3048	meter per second (m/s)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)

The terms "water-surface elevation" and "stage" are used interchangeably in this report, and both are referenced to sea level. River mileage used in this report is taken from U.S. Army Corps of Engineers (1972).

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ABSTRACT

Available data for floods on the Alabama River in the vicinity of Montgomery since the late 1800's indicate that the flood of 1886 is the highest since settlement of the area in 1814.

Profiles for floods in 1948, 1961, 1976, and 1979 were defined using high-water marks and recorded peak stages at gaging stations at Selma and Montgomery. The floods in 1948 and 1961 occurred before construction of Robert F. Henry Lock and Dam (formerly Jones Bluff Lock and Dam). Considerable water-surface differences are indicated between the profiles for periods before and after construction of the dams. Results of an investigation of the reach of river upstream from Selma indicate that the main causes of these differences are clearing and grubbing in preparation for the dams, and operations at the dams. The investigation included a step-backwater model to reproduce profiles based on the actual flood profiles. The profiles were computed for discharges ranging from 80,000 ft^3/s to 500,000 ft^3/s. Stage-discharge relations were then developed for five sites upstream from Selma.

Discharge coefficient curves for prevalent types of flow over the spillway at Robert F. Henry Lock and Dam were developed using discharges obtained from current-meter measurements and standard discharge equations for flow through gate-controlled spillways.

INTRODUCTION

Robert F. Henry Lock and Dam, completed in 1971, and Millers Ferry Lock and Dam, completed in 1969, have resulted in changes to flow characteristics of the Alabama River upstream from the city of Selma. Definition and extent of some of these changes are essential for efficient flood plain management and future development. In order to define these changes, an investigation was conducted for a reach of the river upstream from Selma to Parker Island.

The purpose of this report is to present findings of the investigation using available hydraulic and hydrologic data to address three major topics:

1. Flood profiles for the study reach.
2. Stage-discharge relations computed for five sites within the study reach.
3. Discharge coefficients for the gated spillway section at Robert F. Henry Lock and Dam.

The scope of work includes the assembly of available hydraulic and hydrologic data, and application of a step-backwater model to develop flood profiles and stage-discharge relations. Data from current-meter measurements were used in flow equations to compute discharge coefficients for the spillway section at Robert F. Henry Lock and Dam.

This report was prepared by the U.S. Geological Survey in cooperation with the U.S. Army Corps of Engineers, Mobile District. Appreciation is expressed to the Corps for their assistance.

DESCRIPTION OF THE STUDY REACH

The study reach is the Alabama River between the city of Selma and Parker Island, a distance of about 100 mi (fig. 1). The Alabama River is formed at Parker Island by the confluence of the Tallapoosa and Coosa Rivers. The Alabama River meanders in a generally westerly direction from Montgomery to Selma. The drainage area increases approximately 13 percent with nearly equal contributions from Catoma, Pintlalla, Big Swamp, and Mulberry Creeks. The study reach includes the cities of Montgomery and Selma and the Robert F. Henry Lock and Dam.

Robert F. Henry Lock and Dam, located approximately midway between the cities of Selma and Montgomery, consists of an earthen dam, a gated spillway section, a navigation lock, and a hydroelectric plant. An aerial view of the facility is shown in figure 2. The dam forms R. E. Woodruff Lake which covers a surface area of about 12,500 acres at normal pool elevation, 125 ft above sea level.

The river channel width varies; however, a navigation channel 9 ft deep and 200 ft wide is maintained throughout the study reach. The natural channel is well entrenched with moderate- to steep-sloped banks that before the late 1960's were covered with trees and undergrowth. During the late 1960's, extensive clearing and grubbing of the banks were completed prior to filling the lake.

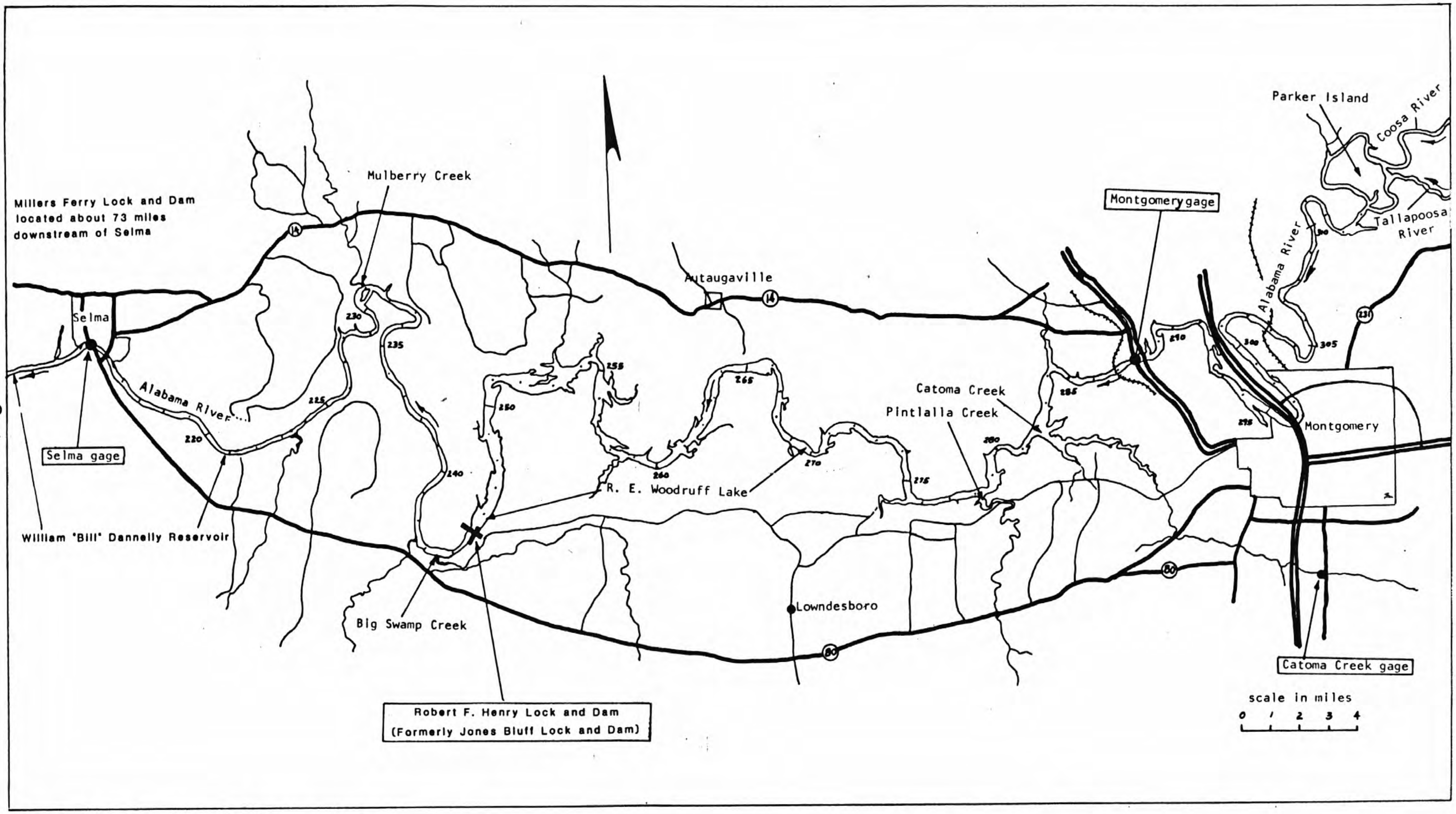
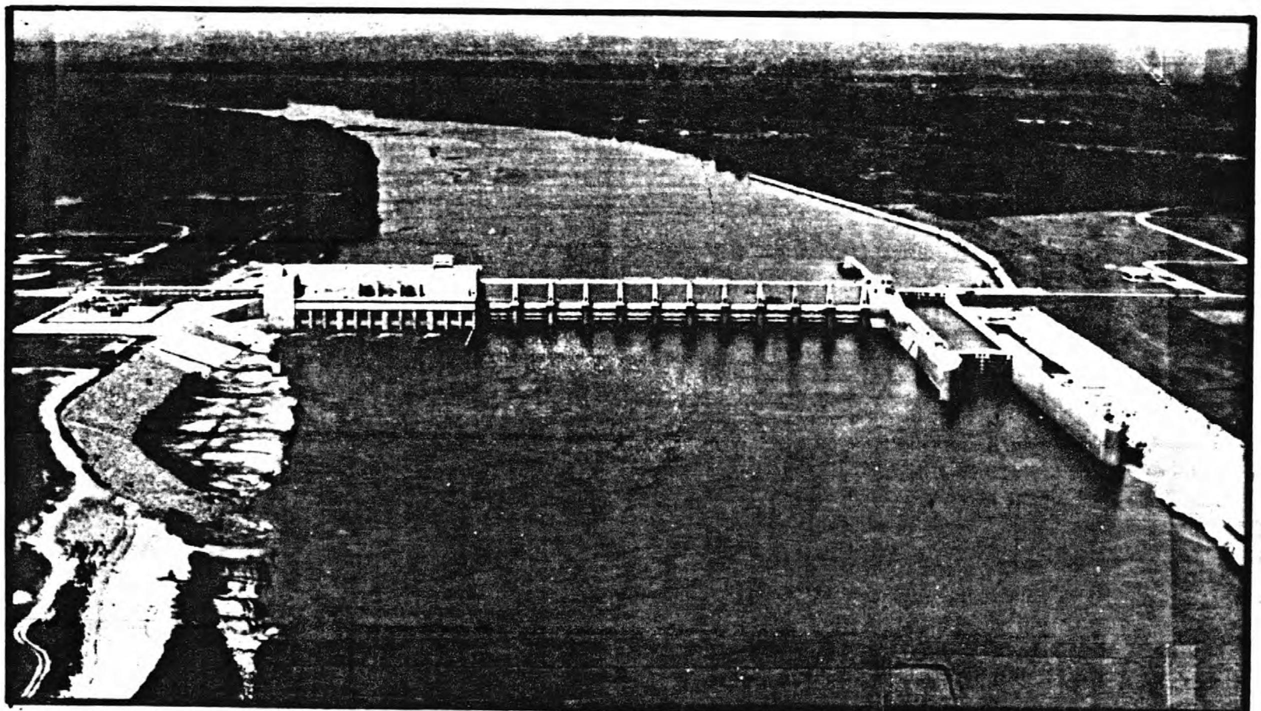


Figure 1.- Study reach and vicinity



Photograph courtesy of U. S. Army Corps of Engineers

Figure 2.- Robert F. Henry Lock and Dam

Downstream from the dam the average channel width is about 600 ft; upstream the channel decreases from about 2,000 ft at the dam to about 500 ft at Parker Island. The flood plain is wide and flat and predominantly wooded with occasional cultivated areas or pasture. Land surface elevations range from about 90 ft at Selma to about 150 ft at Parker Island.

GAGING STATIONS

Two gaging stations have been operated on the Alabama River since the early 1900's. A station at Selma (02423000) (fig. 1) was established in 1900. Daily discharge records for this station are available for the periods 1900 to 1913 and 1928 to 1970. On October 1, 1970, this gage was converted to a "stage only" station and is still in operation (1984). A station at Montgomery (02420000) (fig. 1) was established in 1927 and daily discharge records for this station are available from 1927 to present (1984). Gage-height records are also available for the period 1899 to 1904 at a site 9.3 mi upstream from the present Montgomery gage location.

HISTORICAL FLOODS

Five historical floods are identified in a book on the early history of Montgomery, Alabama by Beale and others (1878). Floods are reported to have occurred in the vicinity of Montgomery in 1822, 1833, 1844, 1855, and 1866. Stages for these floods were not given in the book. However, the 1833 flood was reported to have been the highest since settlement of the area in 1814. Subsequent information indicates that the 1886 flood was higher than the 1833 flood.

Selected flood data for gaging stations at Selma and Montgomery are given in table 1. A complete list of annual peak discharges and stages through 1971 is given in a flood frequency report by Hains (1973). Floods of 1961 and 1979 are documented in reports by Barnes and Somers (1961) and Edelen and others (in publication).

Table 1.--Selected flood data for the Alabama River

Date	Peak stage (in feet)	Peak discharge (ft^3/s)	Remarks
Alabama River at Selma			
1865	112.9**	---	
1874	115.0**	---	
4-08-1886	118.8*	248,000	Discharge estimated
3-19-1929	117.3	204,000	
4-12-1938	117.2	192,000	
12-03-1948	117.8	202,000	
3-01-1961	119.8	284,000	
4-12-1964	115.1	199,000	
4-03-1976	111.6	---	
4-18-1979	116.8	211,000	
Alabama River near Montgomery			
4-01-1886	160.6*	322,000	Discharge estimated
3-30-1888	158.5*	283,000	Discharge estimated
3-17-1929	157.5	256,000	
4-10-1938	154.7	214,000	
12-01-1948	156.1	234,000	
2-26-1961	158.4	283,000	
4-10-1964	152.6	179,000	
4-01-1976	142.6	140,000	
4-16-1979	152.4	260,000	

* Determined from floodmarks near the station.
** Furnished by the U.S. Army Corps of Engineers, Mobile District.

FLOOD PROFILES

Flood profiles were developed for the floods of 1948, 1961, 1976, and 1979 (fig. 3). The floods in 1948 and 1961 occurred prior to construction of Robert F. Henry Lock and Dam. Profiles for these floods are defined from floodmarks collected during or immediately following each flood. Recorded peak sbd/xo db /d/lg/ obdbljgo db Melma and near Montgomery provide additional definition.

In addition to the fact that the 1948 and 1961 floods occurred before the construction of Robert F. Henry Lock and Dam, other differences characterize the floods. The 1948 and 1961 floods differed from the 1976 and 1979 floods by the contrasting types of storm systems producing the floods. The storm systems associated with both the 1948 and 1961 floods were wide spread and produced significantly higher tributary peaks downstream from Montgomery than the 1976 and 1979 floods. The storm systems associated with the 1976 and 1979 floods were concentrated in the upper part of the basin. Figure 4 illustrates hydrographs of selected floods on the Alabama River and Catoma Creek.

COMPUTED PROFILES

Limited observed water-surface profile information often leads to questions that can best be answered through the use of a computerized technique. Water-surface profiles can be computed using a standard step-backwater model described in a report by Shearman (1976).

Profiles were computed for the study reach using the profiles from high-water mark shown in figure 3 and the step-backwater model. Two conditions were selected for model calibration; the first using cross sectional data prior to construction of the Robert F. Henry Lock and Dam and the second after construction. Cross sectional properties were described at 42 sections. Coefficients of roughness (Manning's n) for the channel for each cross section were estimated in the field. Values of Manning's n for the flood plain were estimated using aerial photographs. Elevations of channel sections were determined using fathometer soundings from points of known water surface elevations at each cross section. Ground elevations for the flood plain for each cross section were determined from topographic maps.

To calibrate the model, successive computer runs were made with adjustments to Manning's roughness coefficient after each run until the profiles as defined by high-water marks were reproduced. After calibration other profiles were then generated for both before and after construction at the lock and dam for discharges ranging from 80,000 ft^3/s to 500,000 ft^3/s (figs. 5 and 6).

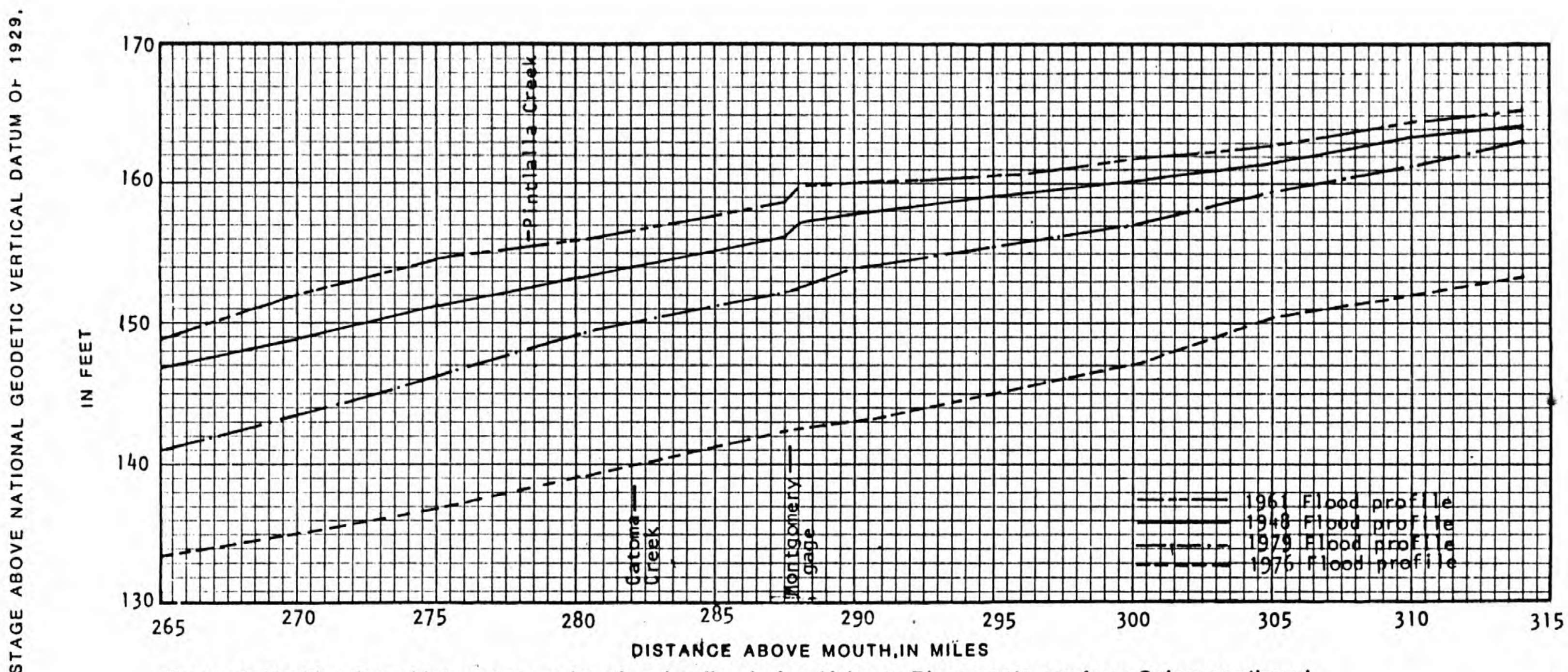


Figure 3.-Profiles from high-water marks of major floods for Alabama River upstream from Selma, continued.

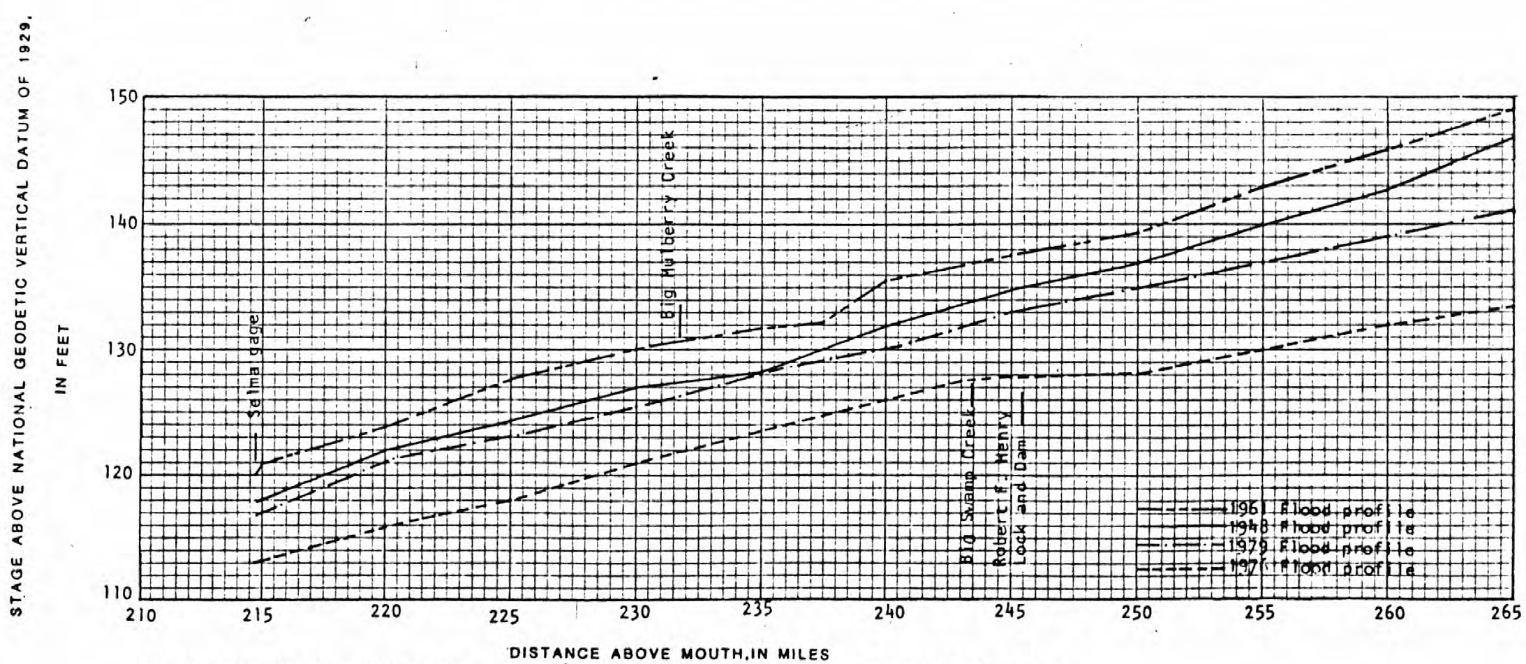


Figure 3.-Profiles from high-water marks of major floods for Alabama River upstream from Selma.

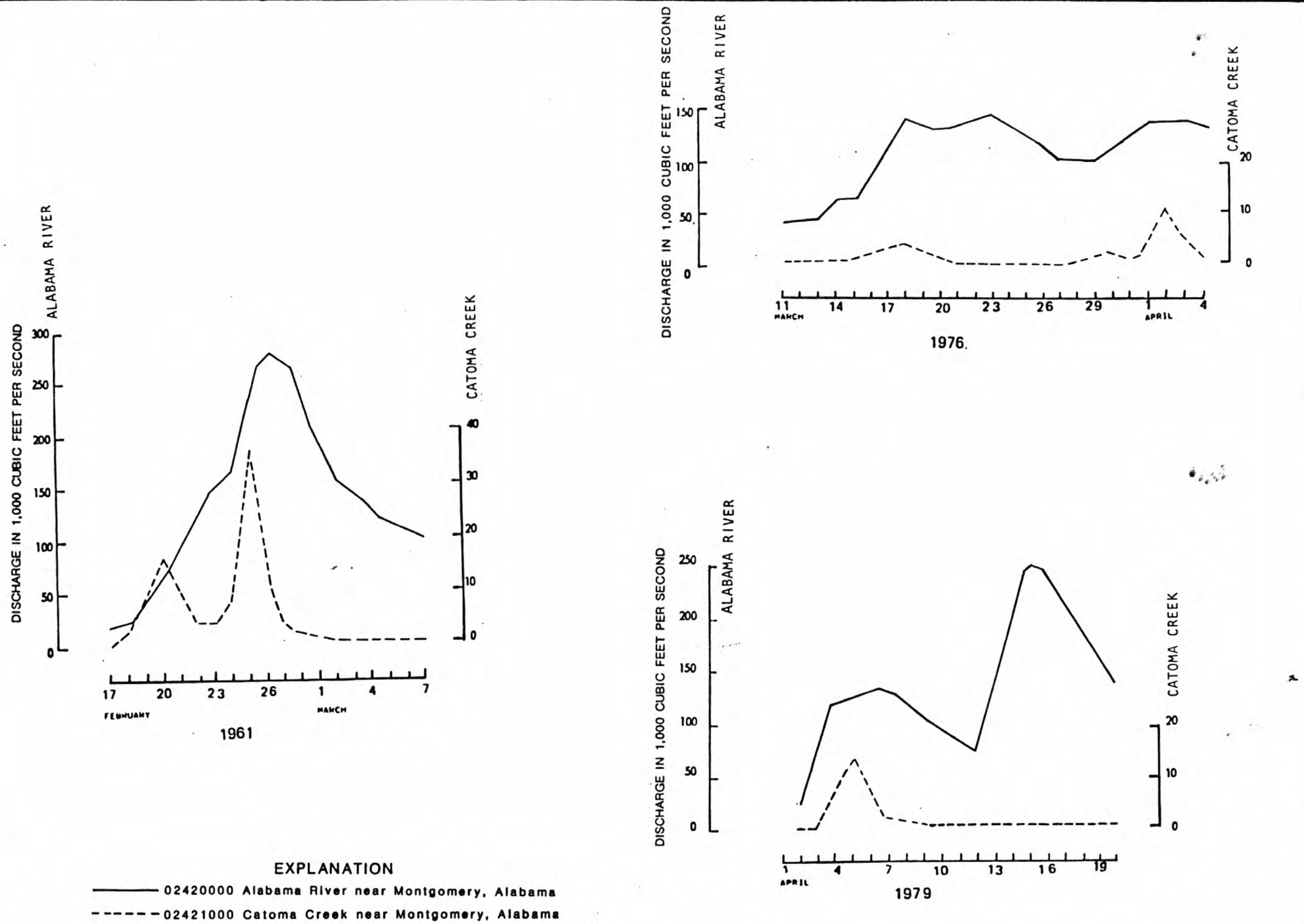


Figure 4.-Hydrographs of discharge for selected floods on Alabama River and Catoma Creek

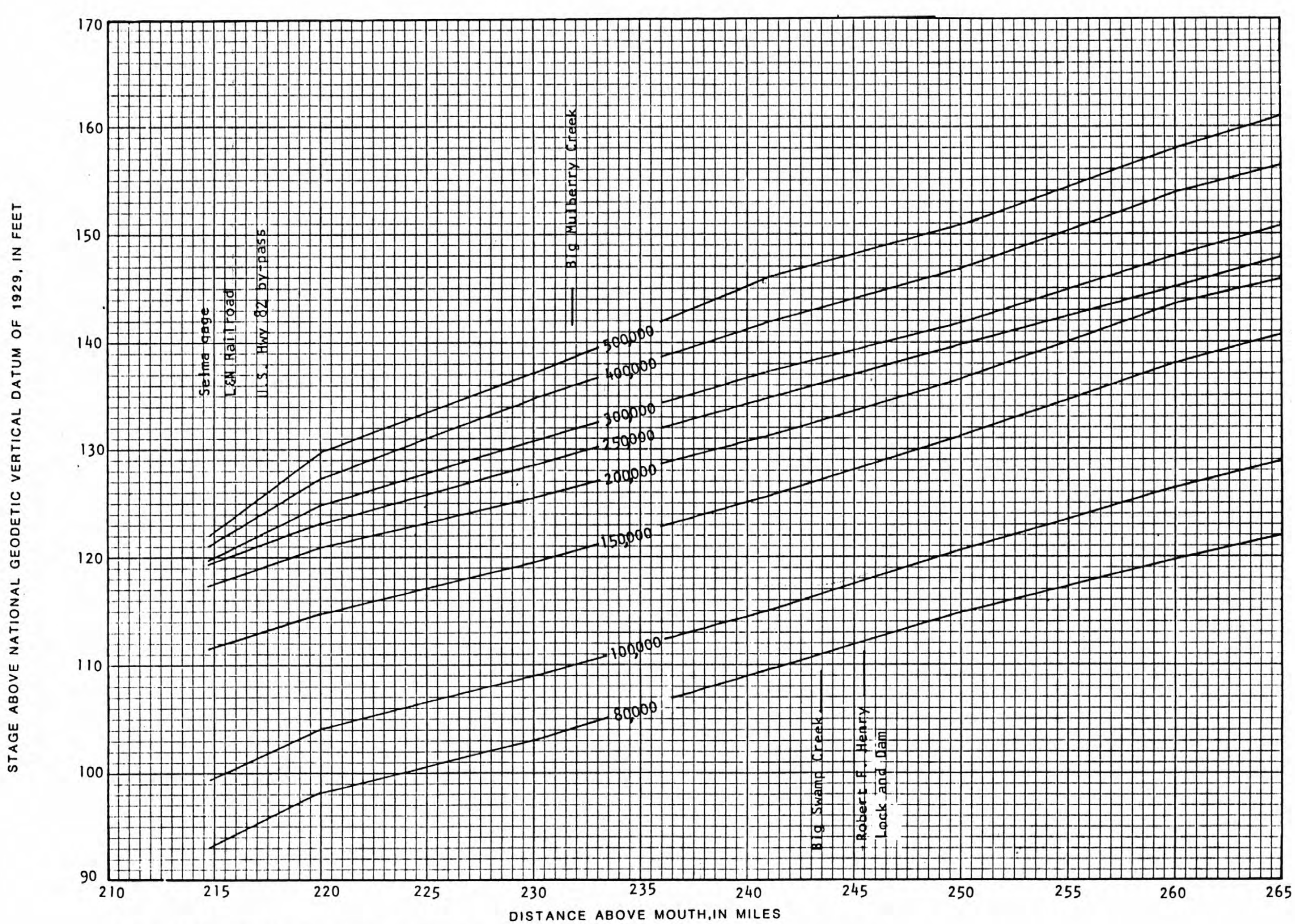


Figure 5.-Step-backwater model profiles before construction of Robert F. Henry Lock and Dam.

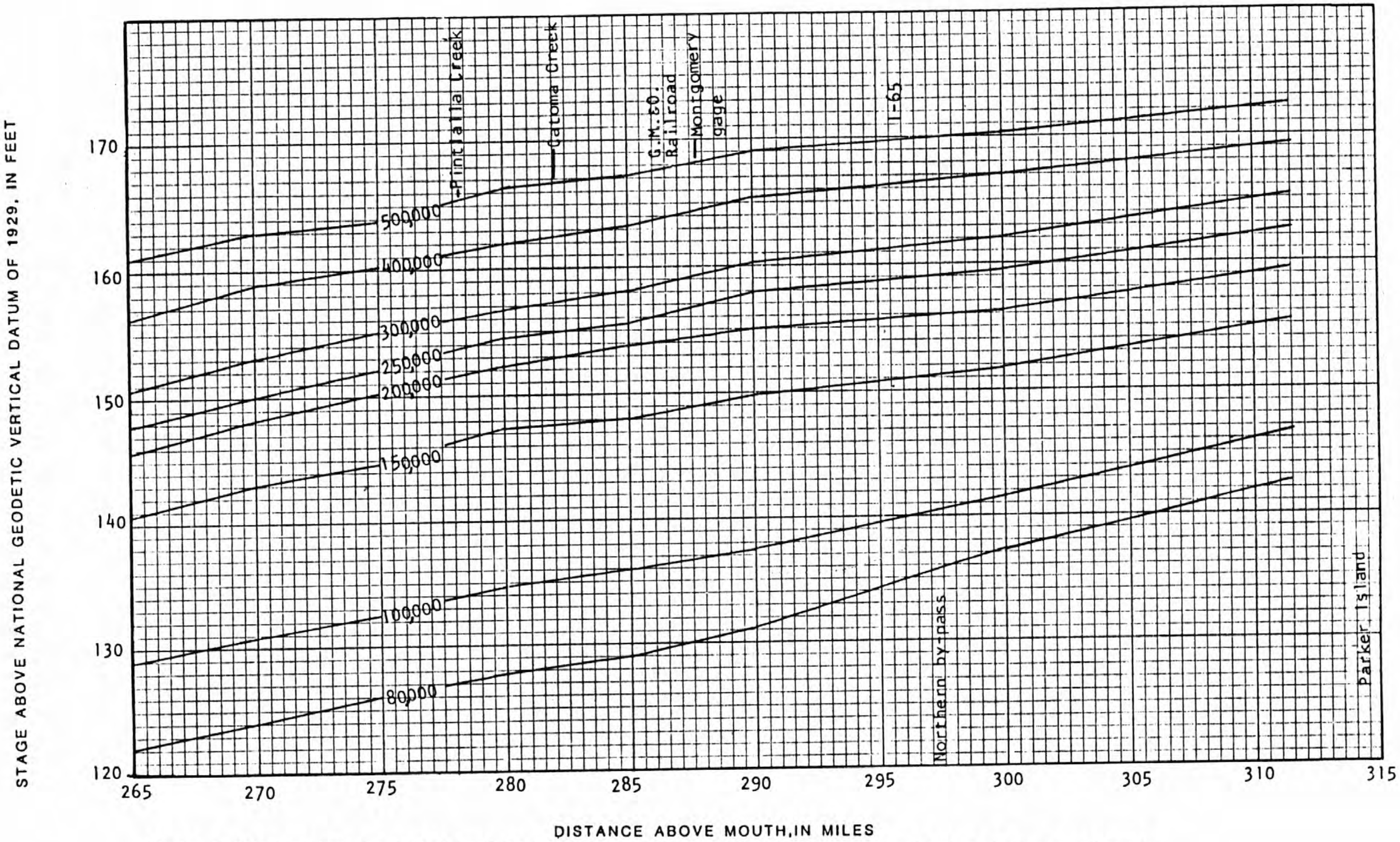


Figure 5.-Step-backwater model profiles before construction of Robert F. Henry Lock and Dam,continued.

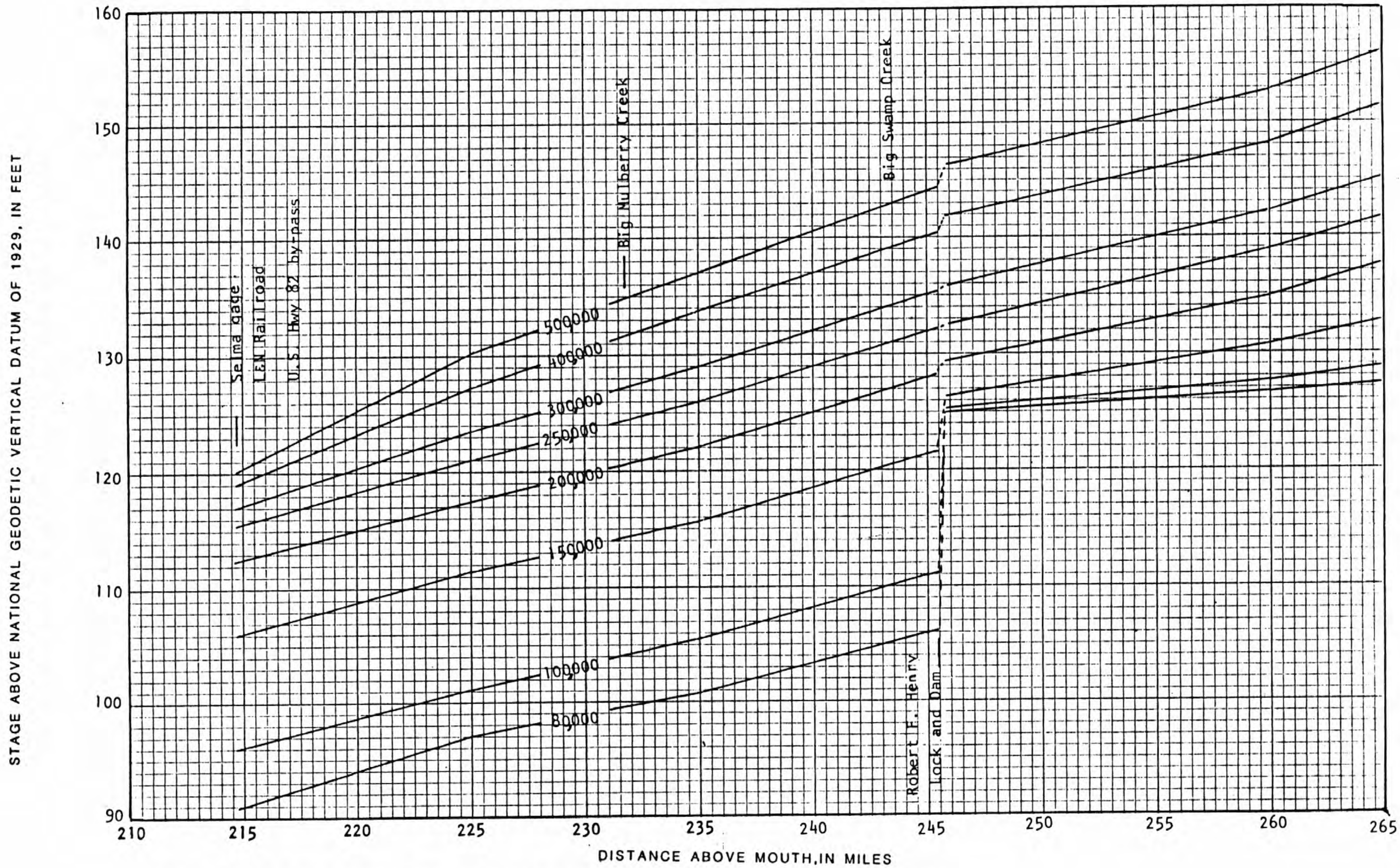


Figure 6.-Step-backwater model profiles after construction of Robert F. Henry Lock and Dam.

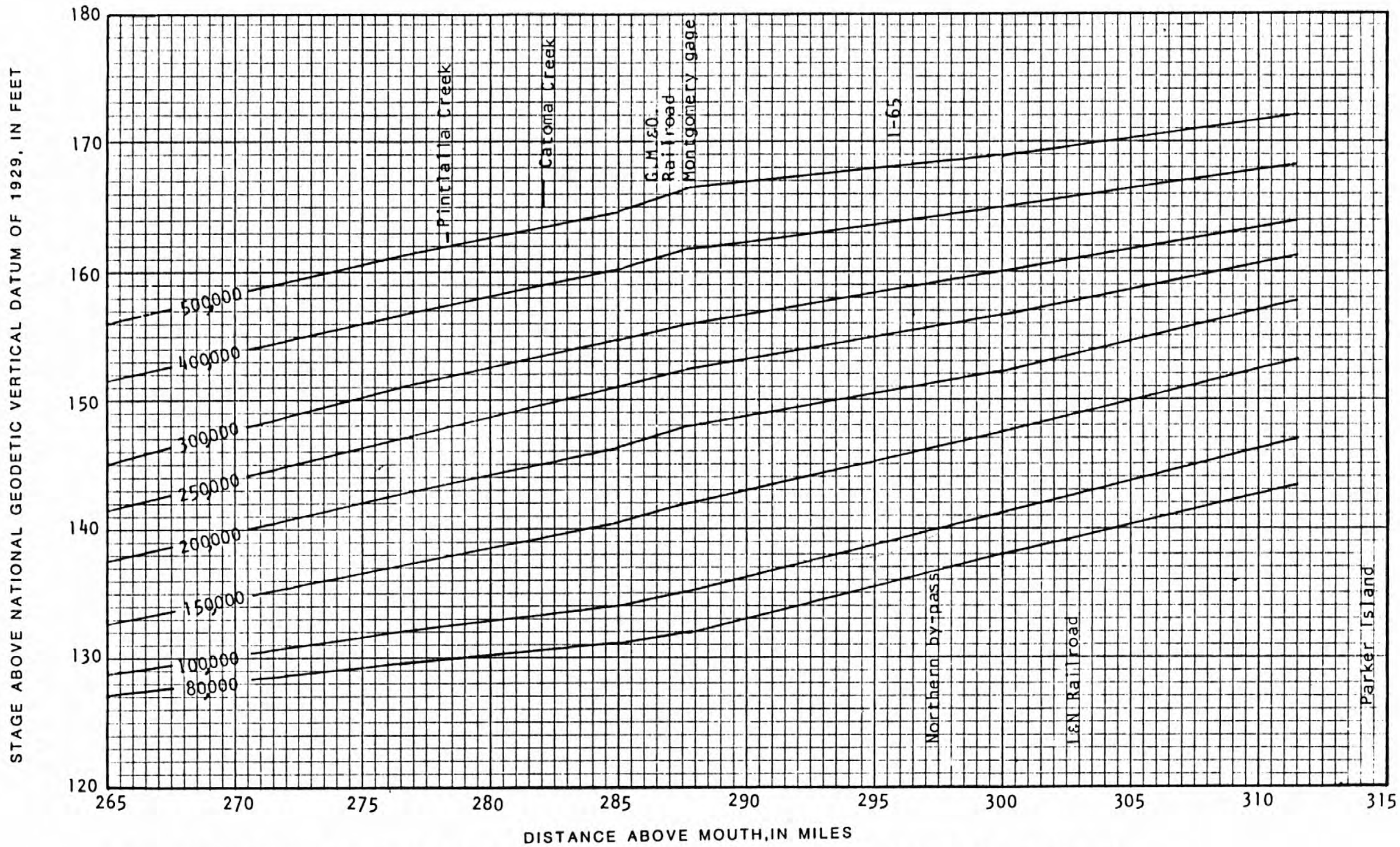


Figure 6.-Step-backwater model profiles after construction of Robert F. Henry Lock and Dam,continued.

STAGE-DISCHARGE RELATIONS

Stage-discharge relations have been developed at five sites within the study area. The sites are: (1) the gaging station at Selma (02423000), (2) the downstream side of Robert F. Henry Lock and Dam (tailwater), (3) the upstream side of Robert F. Henry Lock and Dam (pool), (4) the gaging station near Montgomery (02420000), and (5) just downstream from the south end of Parker Island. The relations are based primarily on the computed flood profiles shown in figures 5 and 6. Additional definition for some of the relations is provided by current-meter measurements. Relations developed using the computed profiles before construction are labeled "B". Those developed using the computed profiles after construction are labeled "A".

Stage-Discharge Relations for Alabama River at Selma (02423000)

Stage-discharge relations A and B (fig. 9) were developed using computed profiles between Millers Ferry Lock and Dam and Selma. These computed profiles are not shown in this report. Current-meter measurements made at the gaging station at Selma were used to better define relation B. The lower end of relation A (stages below about 90 ft) is affected by regulation of flow at Millers Ferry Lock and Dam and, therefore, is not shown below this point.

Stage-Discharge Relations for Robert F. Henry Lock and Dam Tailwater

The low discharge end of relation B was developed using data from current-meter measurements made at the Robert F. Henry Lock and Dam site before construction (fig. 10). The upper end of curve B was defined using the computed profiles shown in figure 5. Relation A was developed using the computed profiles shown in figure 6. Discharge computed through the dam spillway for various stages was also used to better define relation A.

Stage-Discharge Relations for Robert F. Henry Lock and Dam Pool

The lower discharge end of relation B (fig. 11) was developed as an average using data from current-meter measurements made at the dam site before construction. The upper end of the relation (above a stage of 100 ft) was developed using the computed profiles shown in figure 5. Discharge estimates for the 1948 and 1961 floods provide additional definition. Relation A was developed using the data from current-meter measurements and discharge computed through the dam spillway.

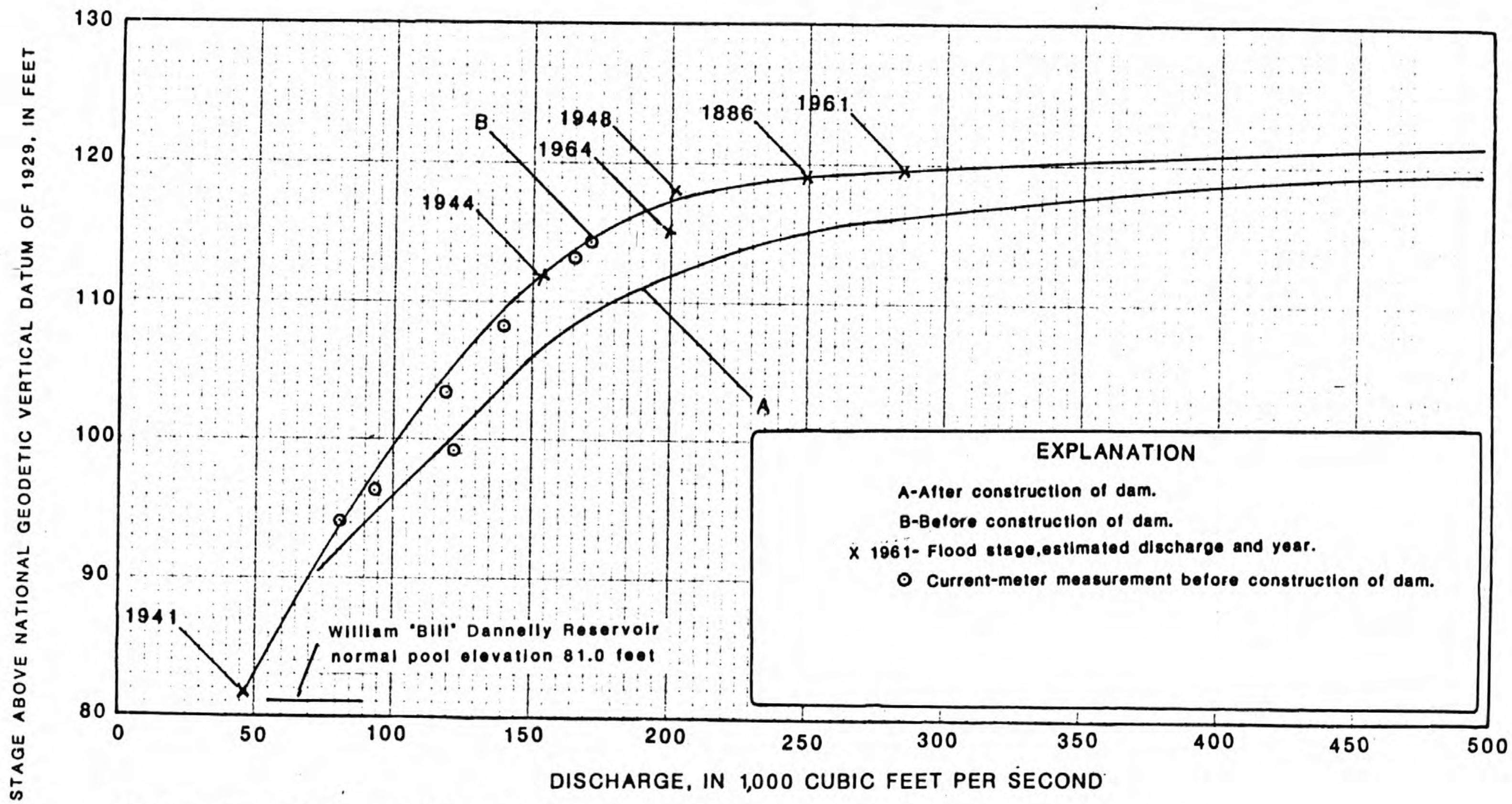


Figure 7.- Stage-discharge relations for the Alabama River at Selma(02423000).

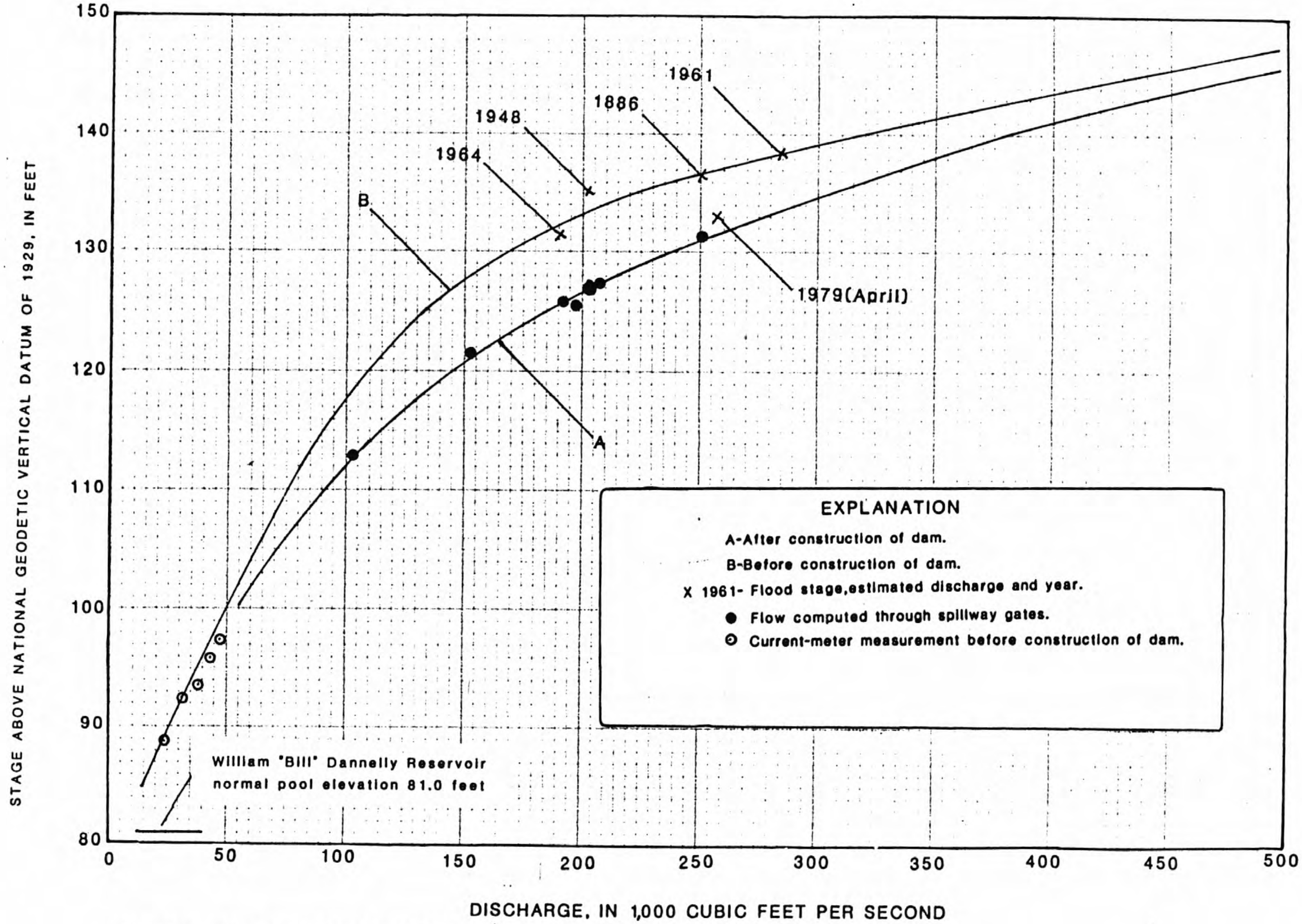


Figure 8.- Stage-discharge relations for Alabama River at Robert F. Henry Lock and Dam tailwater.

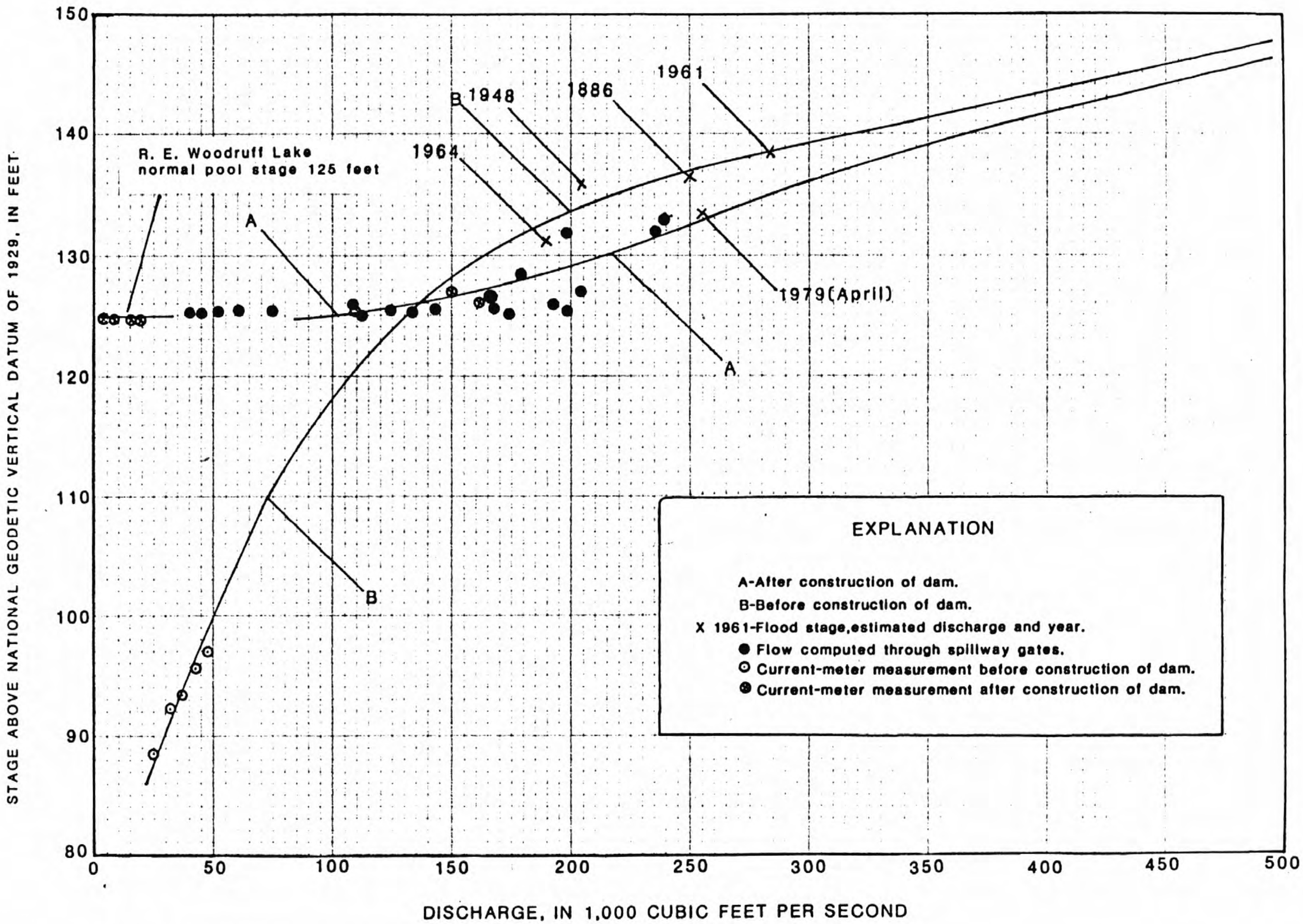


Figure 9.- Stage-discharge relations for Alabama River at Robert F. Henry Lock and Dam.

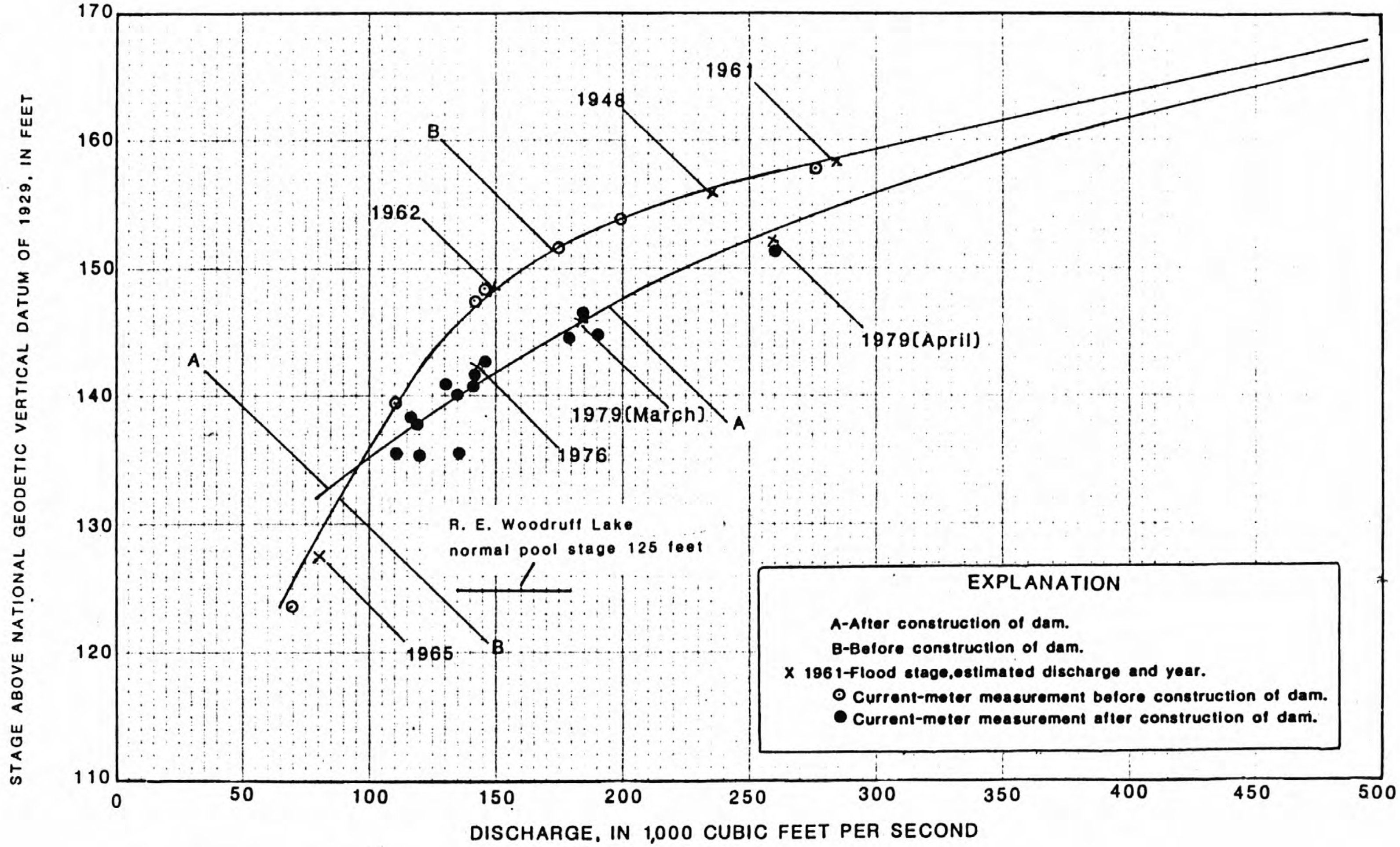


Figure 10.- Stage-discharge relations for Alabama River near montgomery(02420000).

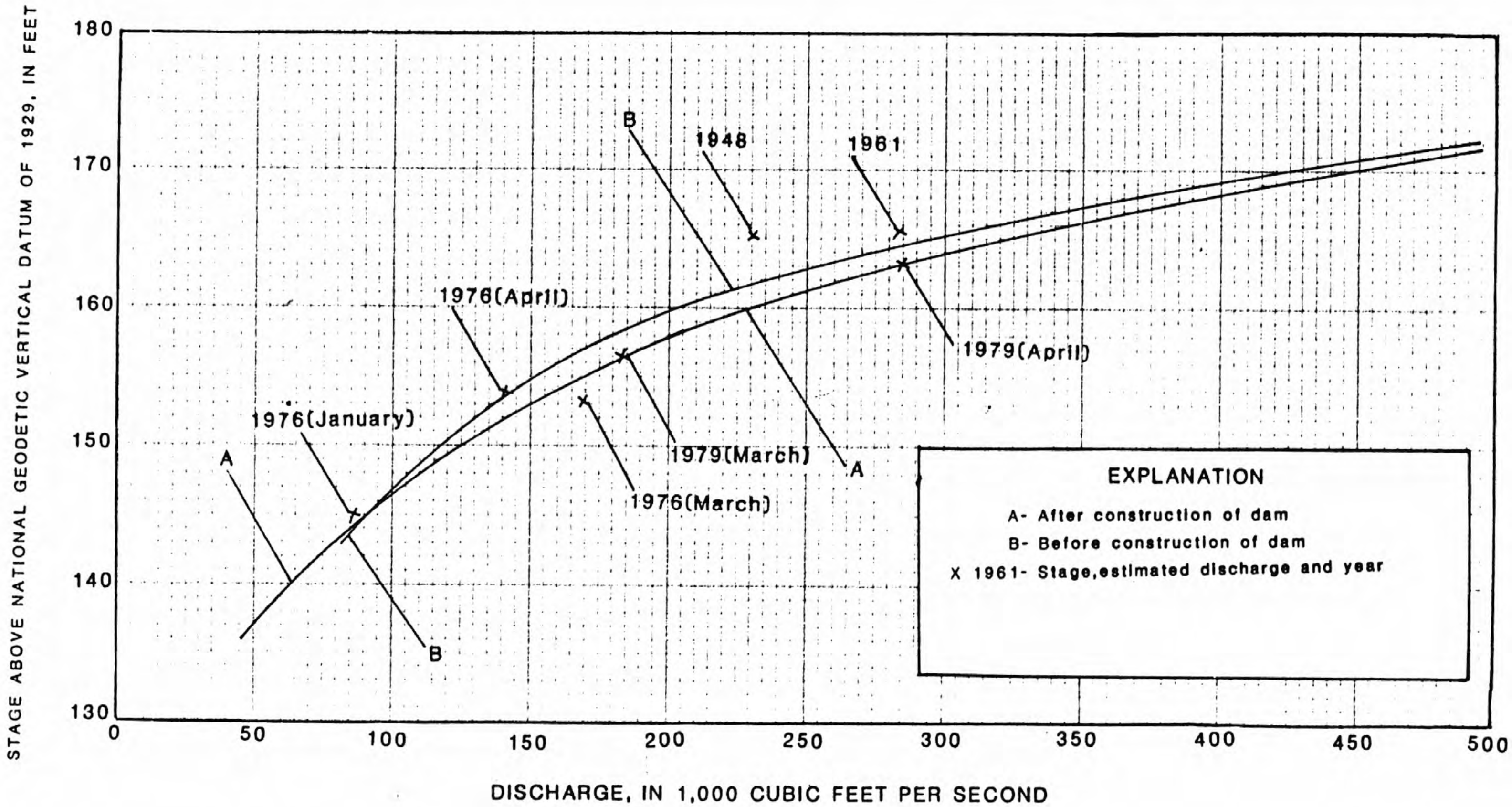


Figure 11.- Stage-discharge relations for Alabama River downstream from Parker Island.

Stage-Discharge Relations for Alabama River near Montgomery (02420000)

Both relations B and A (fig. 12) were developed using the computed profiles shown in figures 5 and 6. Data from current-meter measurements made at the Montgomery gage before construction of the dam were used to better define relation B. Discharges for peaks occurring after construction were used to help define relation A.

Stage-Discharge Relations for the Alabama River Downstream from Parker Island

Both relations A and B (fig. 13) were developed using the computed profiles shown in figures 5 and 6. Estimated discharge for peaks were used to help define the curves.

ENVELOPING CURVES FOR GAGING STATIONS

Twenty-two stage-discharge relations were developed for the Selma station during its operation as a daily-discharge station. Thirty-three stage-discharge relations were developed for the Montgomery station. The relations were defined by more than 500 current-meter measurements made at each site before dam construction.

Curves were drawn that envelop a plot of all the stage-discharge relations for both gaging stations (figs. 14 and 15). They represent conditions before the dam was completed. To avoid clutter only the enveloping curves are shown to a discharge of 140,000 ft^3/s. The enveloping curves are not shown above this discharge because of the lack of definition by current-meter measurements. However, an average curve is shown above 140,000 ft^3/s. This curve is based on all current-meter measurements made at the gaging stations.

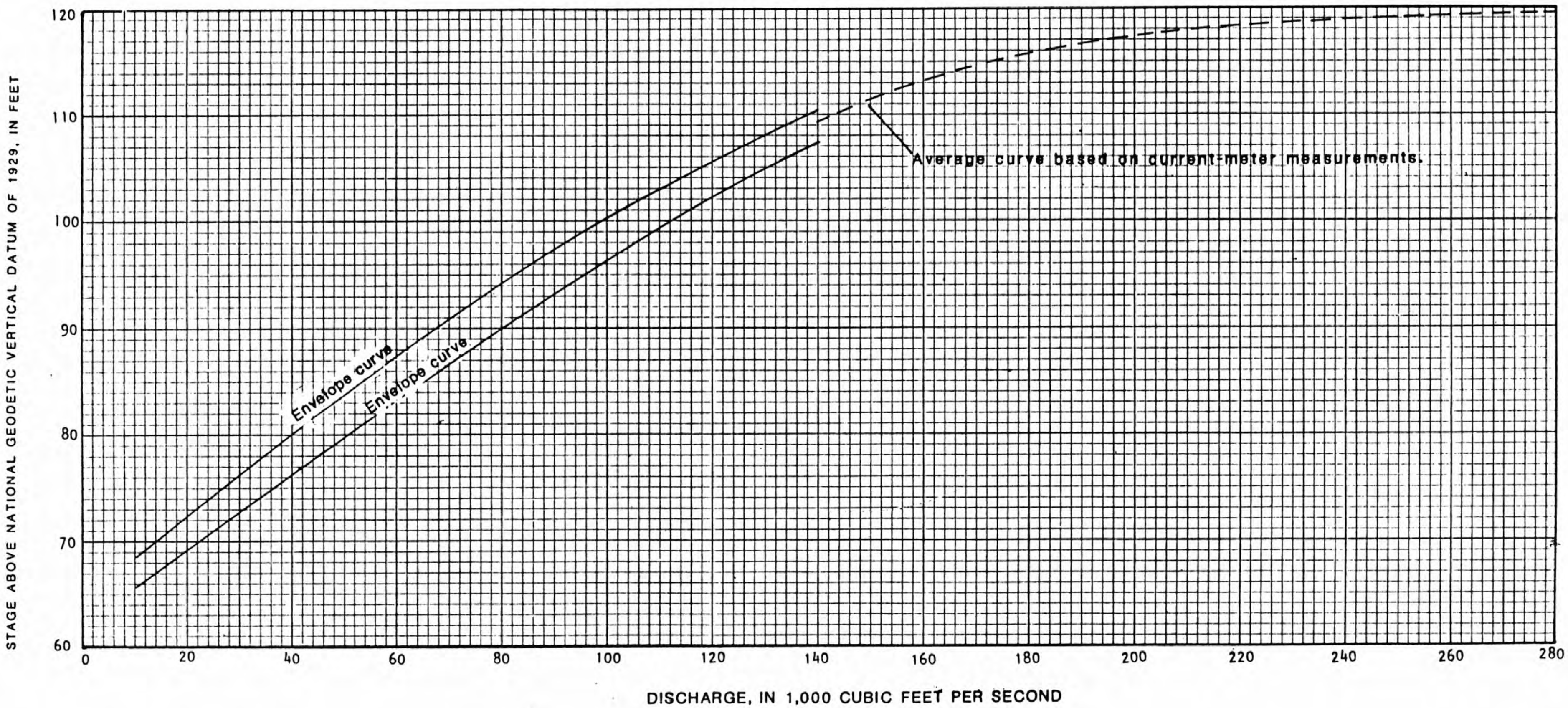


Figure 12.-Enveloping curves for Alabama River at Selma(02423000)

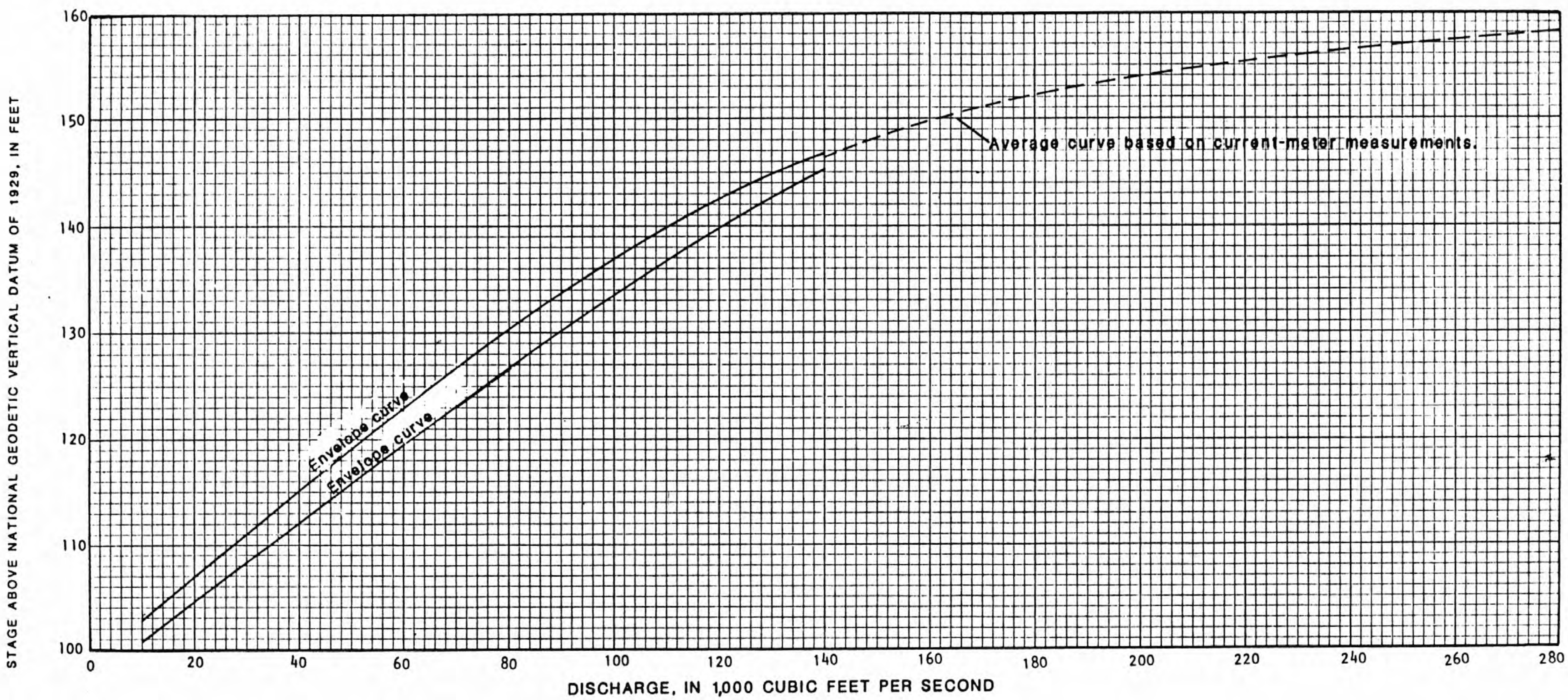


Figure 13.-Enveloping curves for Alabama River near Montgomery(0242000)

Figure 14.- Elevation of flood plain with subsection rating curves at river mile 247.51

Figure 15.- Elevation of flood plain with subsection rating curves at river mile 264.33.

CROSS-SECTIONAL CHANGES

Cross sections are shown for four sites that are typical of the river channel for conditions before and after construction of the dam (figs. 16-19). These cross sections were selected from those used in the step-backwater study. The cross sections are divided into subsections based on section shape and on estimates of roughness.

Stage-discharge relations were estimated for each subsection. These relations were estimated based on a distribution of discharge for a known water surface elevation. Discharges and elevations used in the distribution were taken from the results of the step-backwater study for each of the cross sections. The distribution was determined by proportioning discharge to each subsection based on the respective subsection conveyance. Conveyance is the capacity of the subsection to transport water based on its area and related roughness.

CHANNEL VELOCITIES

Mean velocities in the channel were computed for three sites [Alabama River at Selma (02423000), Alabama River at Robert F. Henry Lock and Dam (tailwater), and Alabama River near Montgomery (02420000)] for periods before and after construction. These mean velocities were computed using data from current-meter measurements made at each site (tables 2, 3, and 4). The channel part of each measurement was separated from the flood plain sections and used to compute a mean channel velocity. Plots of mean velocity versus total discharge are given (figs. 20, 21, and 22). To avoid clutter some of the data points were not plotted.

Figure 16.- Elevation of flood plain with subsection rating curves at river mile 280.91

25

Figure 17.- Elevation of flood plain with subsection rating curves at river mile 308.0

EXPLANATION

- o Computed using current-meter measurements made before construction of Robert F. Henry Lock and Dam
- • Computed using current-meter measurements made after construction of Robert F. Henry Lock and Dam

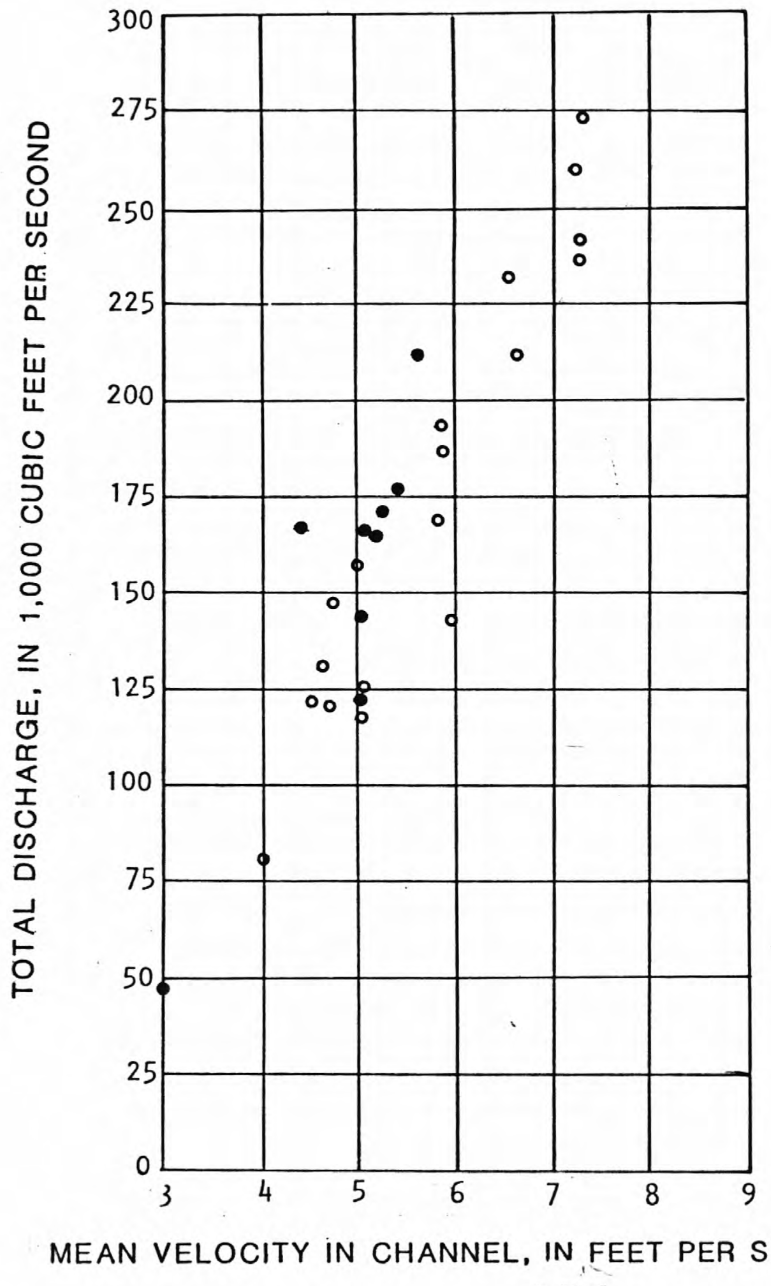


Figure 18.- Discharge-velocity relation for Alabama River at Selma(02423000).

EXPLANATION

- o Computed using current-meter measurements made before construction of Robert F. Henry Lock and Dam
- • Computed using current-meter measurements made after construction of Robert F. Henry Lock and Dam

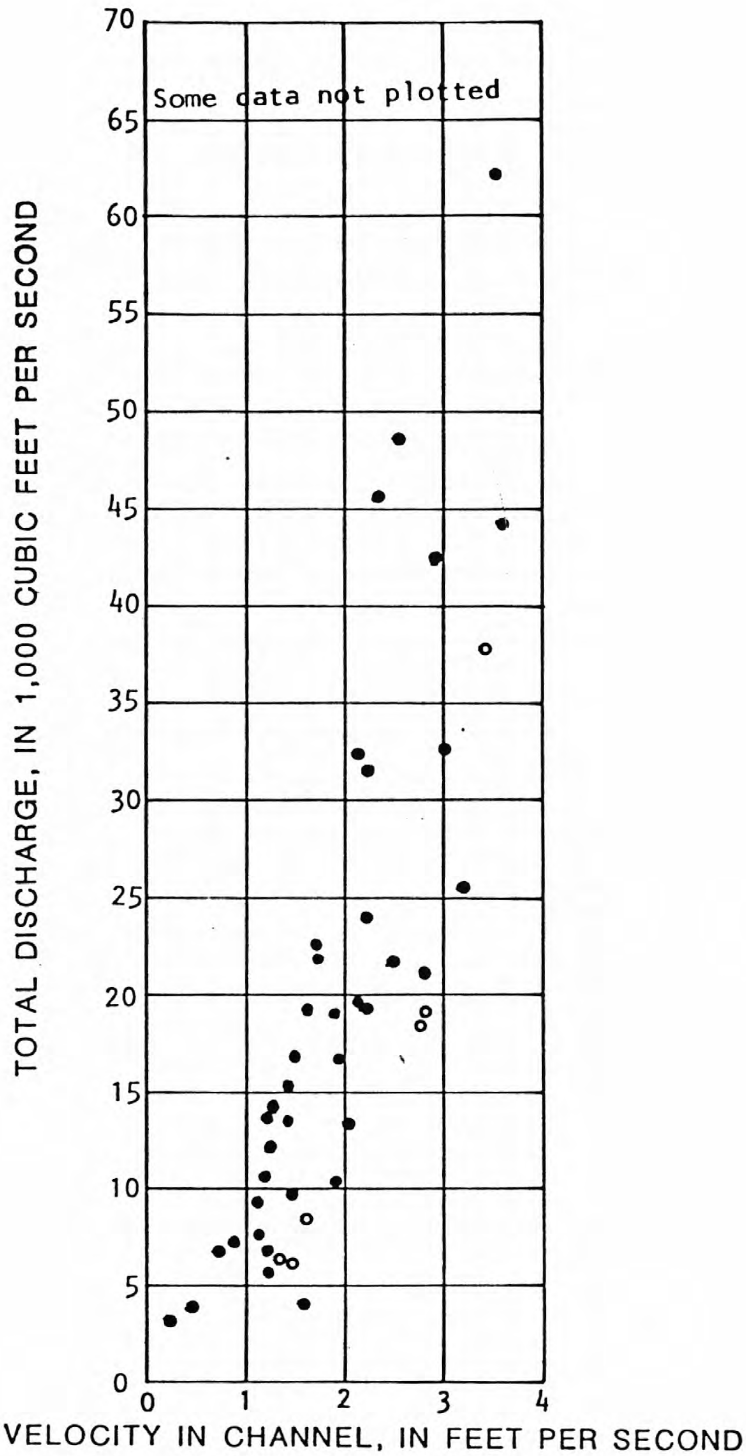


Figure 19.- Discharge-velocity relation for Alabama River at Robert F. Henry Lock and Dam tailwater.

Table 2. Data used to compute mean velocity in the Alabama River channel at Selma (02423000)

Discharge measurement no.	Date	Stage (ft)	Discharge (ft^3/s) Total	Discharge (ft^3/s) Channel	Mean velocity in channel (ft/s)
			Before construction of Robert F. Henry Lock and Dam		
373	2/17/56	100.70	116,000	116,000	5.06
374	3/18/56	103.76	126,000	126,000	5.10
375	3/18/56	104.34	128,000	127,000	5.12
441	2/25/61	115.50	166,000	160,000	5.30
442	2/27/61	118.90	259,000	235,000	7.25
443	2/01/61	119.70	273,000	242,000	7.30
444	3/03/61	118.30	231,000	211,000	6.59
445	3/06/61	113.70	156,000	153,000	5.00
446	3/08/61	111.00	142,000	129,000	6.18
447	3/10/61	108.60	130,000	129,000	4.66
457	12/01/61	111.97	147,000	146,000	4.85
463	4/17/62	105.68	121,000	120,000	4.55
496	3/19/64	103.40	119,000	118,000	4.76
497	3/25/64	94.14	81,500	81,500	4.01
498	4/09/64	111.83	169,000	165,000	5.75
499	4/10/64	113.75	176,000	172,000	5.64
500	4/12/64	115.11	193,000	187,000	5.90
501	4/14/64	114.29	172,000	167,000	5.48
502	4/17/64	113.43	166,000	162,000	5.33
			After construction of Robert F. Henry Lock and Dam		
573	1/22/73		46,900	46,000	2.97
5bg	vB25/76	99.30	122,000	117,000	5.03
575	4/03/76	111.52	176,000	90,100	5.43
576	3/06/79	111.22	167,000	158,000	4.45
557	3/07/79	112.74	171,000	161,000	5.30
578	3/08/79	112.54	165,000	155,000	5.23
579	3/09/79	111.32	166,000	152,000	5.14
580	3/10/79	109.04	144,000	129,000	5.09
581	4/18/79	116.70	211,000	176,000	5.65

Table 3. Data used to compute mean velocity in the Alabama River channel at Robert F. Henry Lock and Dam

Discharge measurement no.	Date	Stage (ft)	Discharge (ft³/s) Total	Discharge (ft³/s) Channel	Mean velocity in channel (ft/s)
		Before construction of Robert F. Henry Lock and Dam			
1	8/02/66	79.16	6,150	6,150	1.52
2	8/02/66	79.11	6,340	6,340	1.36
3	8/04/66	79.61	6,850	6,850	1.62
4	7/06/67	85.53	19,000	19,000	2.82
5	7/06/67	85.30	18,500	18,500	2.80
6	4/17/68	93.43	37,900	37,900	3.41
		During and after construction of Robert F. Henry Lock and Dam			
8	4/23/70	91.24	32,600	32,600	3.02
9	5/27/70	81.18	5,720	5,720	1.27
10	6/17/70	83.89	13,300	13,300	2.03
11	7/24/70	82.32	10,300	10,300	1.99
12	8/25/70	81.48	6,880	6,880	1.21
13	9/29/70	81.02	6,380	6,380	1.33
14	11/18/70	86.95	21,100	21,100	2.84
15	12/16/70	88.32	25,600	25,600	3.27
16	1/17/71	95.75	44,300	44,300	3.60
17	2/18/71	92.19	32,200	32,200	2.15
18	3/23/71	97.06	48,400	48,400	2.56
19	5/11/71	87.56	21,900	21,900	1.75
20	6/24/71	85.26	15,300	15,300	1.44
21	7/22/71	84.92	14,200	14,200	1.31
22	8/26/71	85.64	16,800	16,800	1.53
23	10/20/71	81.58	7,210	7,210	.88
24	11/05/71	83.91	12,300	12,300	1.24
25	11/17/71	85.96	19,000	19,000	1.70
26	11/23/71	80.68	3,940	3,940	.49
27	2/16/72	95.85	45,500	45,500	2.38
28	3/16/72	89.27	19,200	19,200	1.63
29	4/20/72	86.46	14,000	14,000	1.61
30	5/23/72	83.31	6,950	6,950	.75
31	6/27/72	86.16	13,900	13,900	1.25
32	8/22/72	83.55	13,500	13,500	1.43
33	10/11/72	82.88	10,700	10,700	1.20
34	11/02/72	85.18	24,100	24,100	2.25
35	1/31/73	95.34	42,700	42,700	2.94
36	3/07/73	91.47	31,600	31,600	2.27
37	4/24/73	88.54	22,600	22,600	1.77
38	5/23/73	99.89	61,200	61,200	3.52
39	7/21/73	88.60	19,700	19,700	2.15
40	7/24/73	86.90	16,600	16,600	1.98
41	7/26/73	87.50	21,700	21,700	2.50
42	10/15/73	81.90	7,670	7,670	1.19
43	10/15/73	80.90	3,150	3,150	.43
44	10/16/73	81.50	9,860	9,860	1.48
45	1/23/75	88.40	19,200	19,200	2.25
46	1/24/75	90.80	26,600	26,600	2.77
47	7/01/75	84.94	9,060	9,060	1.16

Table 4. Data used to compute mean velocity in the Alabama River channel near Montgomery (02420000)

Discharge measurement no.	Date	Stage (ft)	Discharge (ft^3/s) Total	Channel	Mean velocity in channel (ft/s)
			Before construction of Robert F. Henry Lock and Dam		
359	7/26/60	99.36	6,280	3,270	1.32
360	9/06/60	99.10	5,830	2,900	1.16
361	11/22/60	99.55	7,140	3,800	1.45
362	1/24/61	102.08	11,400	6,170	1.89
363	2/25/61	154.04	199,000	192,000	5.27
364	2/26/61	157.70	276,000	218,000	5.70
365	3/02/61	152.50	150,000	122,000	3.91
366	3/03/61	150.14	137,000	111,000	3.66
367	3/06/61	140.78	101,000	100,000	3.72
368	3/07/61	140.00	97,700	96,800	3.60
369	3/09/61	139.52	110,000	105,000	4.06
370	3/10/61	138.37	104,000	99,000	3.94
371	3/12/61	133.62	93,000	89,700	3.97
372	5/18/61	103.92	14,100	5,920	1.76
373	7/12/61	104.08	16,000	6,660	2.00
374	9/21/61	101.78	10,800	4,950	1.65
375	11/10/61	99.82	6,770	3,080	1.32
376	12/20/61	148.50	146,200	143,000	4.81
377	2/14/62	109.65	29,900	12,900	2.67
378	4/10/62	114.23	39,900	17,200	2.87
379	4/16/62	140.63	114,000	93,200	5.01
380	6/28/62	102.22	9,610	9,460	1.34
382	12/17/62	99.93	6,940	3,280	1.23
383	1/31/63	109.53	28,700	12,200	2.56
384	3/08/63	126.15	75,600	56,400	4.23
385	3/15/63	130.15	93,700	75,600	4.79
386	5/02/63	129.88	105,000	83,800	5.25
387	7/10/63	104.92	16,200	7,480	1.98
388	9/26/63	100.69	8,320	4,130	1.47
389	12/05/63	104.26	16,400	7,780	2.15
390	1/29/64	123.31	65,500	27,200	3.15
391	3/18/64	138.35	117,000	111,000	4.38
392	4/09/64	151.69	175,000	117,000	5.21
393	5/27/64	107.73	20,900	8,610	1.94
394	6/10/64	104.67	13,500	5,940	1.75
395	8/04/64	101.94	9,040	4,340	1.40
396	9/24/64	101.37	8,900	4,180	1.40
397	12/08/64	108.02	26,100	11,700	2.92
398	2/18/65	124.96	62,800	23,700	2.69
399	5/12/65	102.78	10,400	4,960	1.55
400	7/13/65	102.18	10,500	4,490	1.48
401	9/08/65	100.62	8,620	3,960	1.46

Table 4. Data used to compute mean velocity in the Alabama River channel near Montgomery (02420000) (continued)

Discharge measurement no.	Date	Stage (ft)	Discharge (ft³/s) Total	Discharge (ft³/s) Channel	Mean velocity in channel (ft/s)
			After construction of Robert F. Henry Lock and Dam		
502	1/14/76	127.98	43,900	43,900	2.37
503	2/24/76	127.40	34,200	33,900	1.93
504	3/17/76	140.94	140,000	117,000	4.75
505	3/18/76	140.97	113,000	113,000	4.43
506	3/23/76	135.71	131,000	130,000	5.80
507	3/25/76	132.78	100,000	99,000	4.67
508	4/01/76	141.60	142,000	136,000	6.00
509	4/01/76	120.67	145,000	138,000	5.23
510	4/06/76	135.35	120,000	118,000	5.21
511	4/20/76	125.71	17,900	16,700	0.96
512	5/28/76	125.46	31,900	30,700	1.76
513	7/02/76	126.50	25,800	24,900	1.41
514	7/26/76	125.32	15,600	14,500	0.85
515	8/16/76	125.57	3,110	2,900	0.21
516	9/21/76	125.70	6,300	6,300	0.37
517	10/22/76	125.54	18,800	18,800	1.05
518	11/19/76	124.93	16,700	16,700	0.99
519	12/17/76	125.64	28,900	28,500	1.64
520	2/03/77	124.76	22,900	22,900	1.36
521	4/01/77	141.38	137,000	136,000	5.35
522	4/04/77	139.72	140,000	138,000	5.16
523	4/17/77	139.16	140,600	140,200	4.95
524	5/11/77	125.98	24,200	24,100	1.30
525	5/26/77	124.08	15,400	15,400	0.75
526	8/16/77	124.08	14,500	14,500	0.73
527	9/27/77	128.28	50,900	47,600	2.39
528	10/28/77	127.21	40,500	38,200	2.55
529	11/07/77	128.64	50,100	48,900	2.47
530	11/09/77	129.03	59,000	58,800	2.91
531	11/11/77	127.88	50,700	50,600	2.54
532	11/15/77	126.94	37,800	37,700	2.01
533	11/25/77	126.37	28,100	27,100	1.45
534	11/29/77	126.19	24,100	24,100	1.31
535	11/30/77	126.98	35,600	35,600	1.87
536	12/02/77	126.31	28,500	27,000	1.46
537	1/27/78	138.39	113,000	113,000	4.56
539	1/29/78	135.72	117,700	117,400	4.98
540	1/30/78	132.32	95,300	94,300	4.28
541	4/19/78	126.50	19,900	19,900	1.04
542	5/10/78	136.11	105,000	105,000	4.49
543	3/05/79	144.67	178,000	174,000	6.23

Table 4. Data used to compute mean velocity in the Alabama River channel near Montgomery (02420000) (continued)

Discharge measurement no.	Date	Stage (ft)	Discharge (ft^3/s) Total	Channel	Mean velocity in channel (ft/s)
			After construction of Robert F. Henry Lock and Dam (continued)		
544	3/06/79	146.29	184,000	166,000	4.86
545	3/07/79		142,000	136,000	4.87
546	4/04/79	138.55	128,000	127,000	5.07
549	4/14/79	144.76	190,000	184,000	6.63
550	4/15/79	151.50	261,000	241,000	7.76
552	4/19/79	145.77	146,000	141,000	4.84
553	7/27/79	127.95	50,000	50,000	2.28
554	12/12/79	125.65	27,500	27,500	1.41
555	1/18/80	127.84	49,400	49,300	2.35
556	2/29/80	125.64	28,700	28,700	1.42
557	3/27/80	136.09	113,000	110,000	4.58
558	5/15/80	125.91	26,900	25,500	1.27
560	9/29/80	125.90	24,400	24,400	1.25
561	12/16/80	124.97	29,700	29,700	1.54
562	2/09/81	124.10	21,600	21,500	1.15

EXPLANATION

- o Computed using current-meter measurements made before construction of Robert F. Henry Lock and Dam
- • Computed using current-meter measurements made after construction of Robert F. Henry Lock and Dam

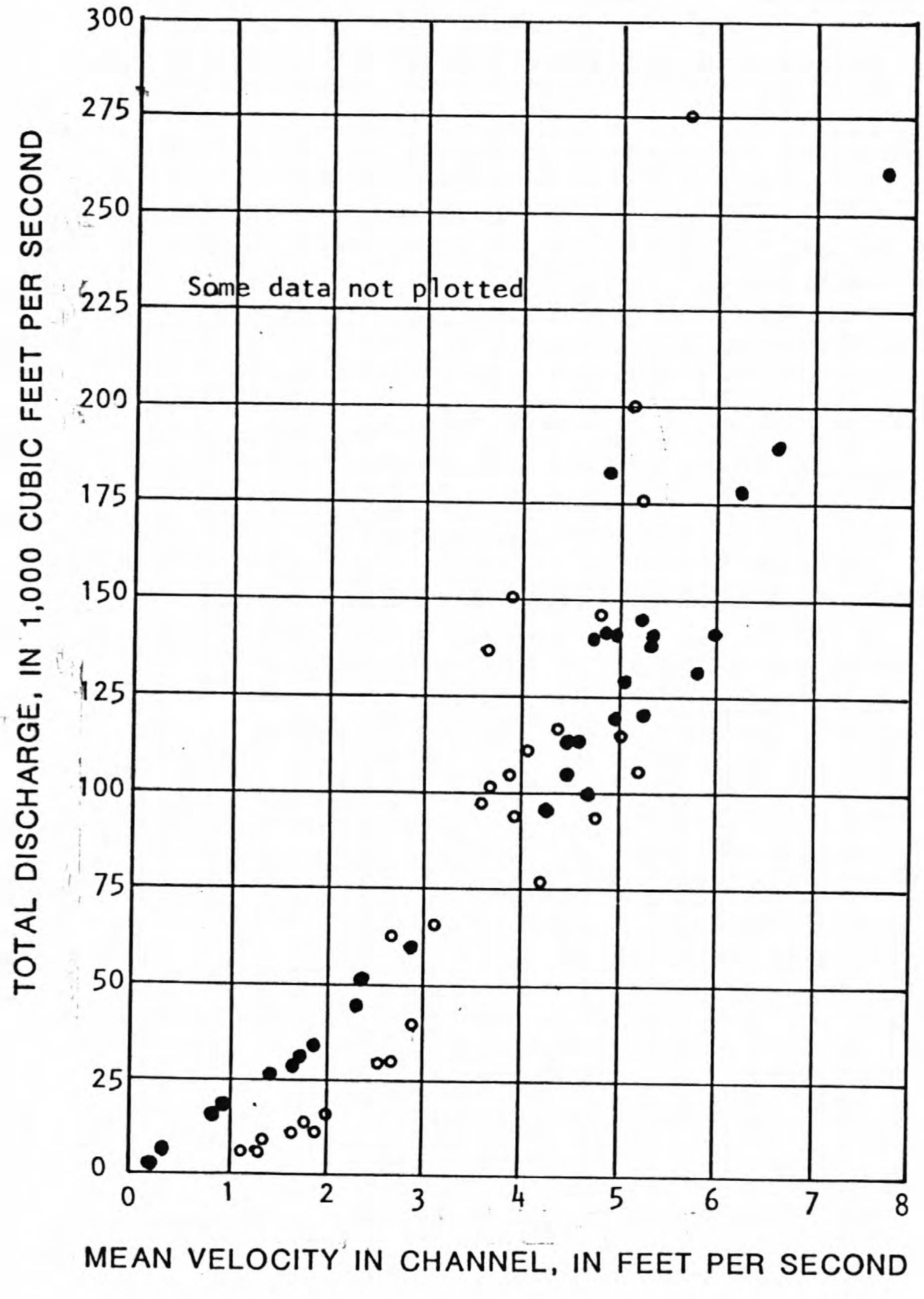


Figure 20.- Discharge-velocity relation for Alabama River near Montgomery (024200).

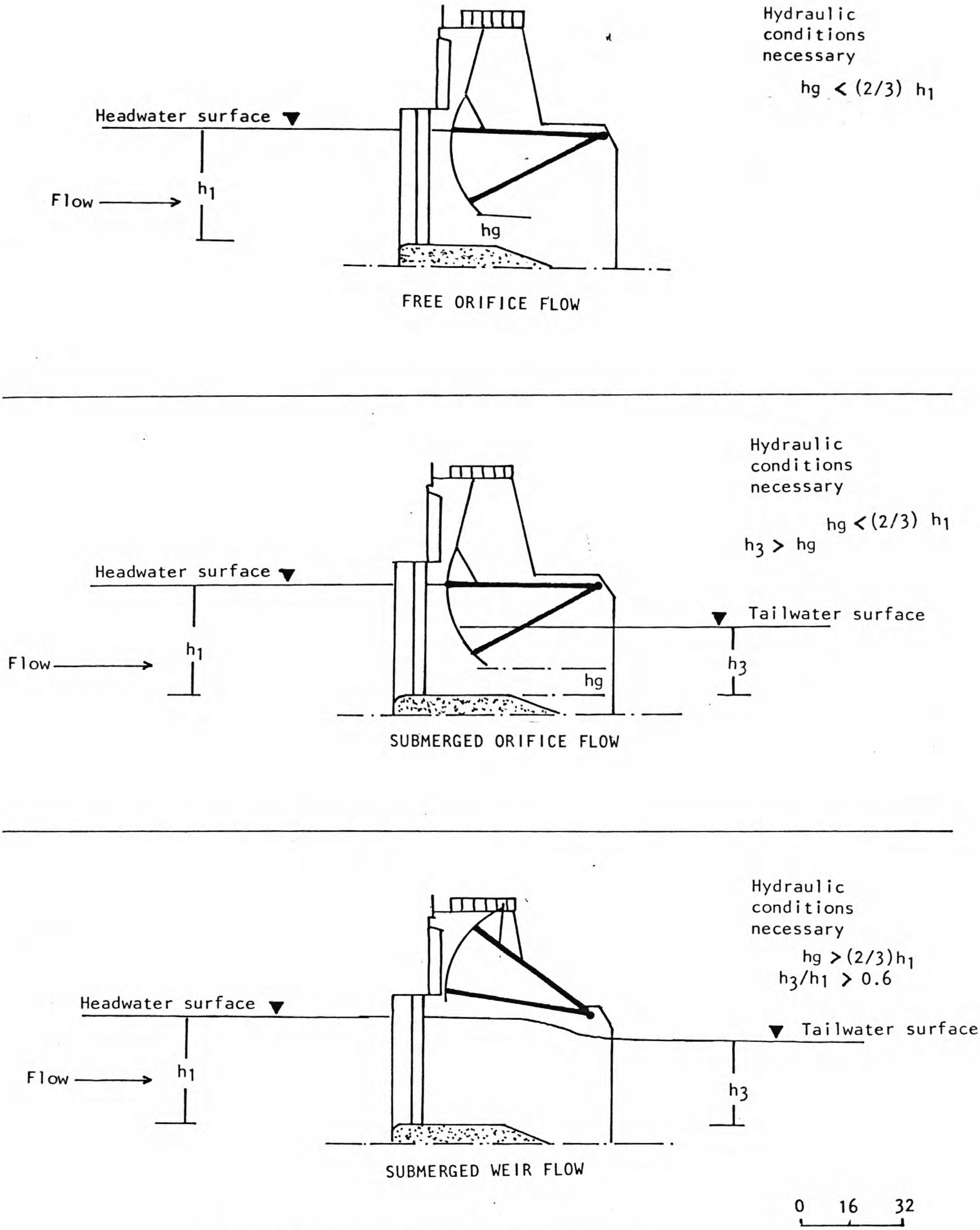


Figure 21.- Definition of orifice and weir flows at Robert F. Henry Lock and Dam.

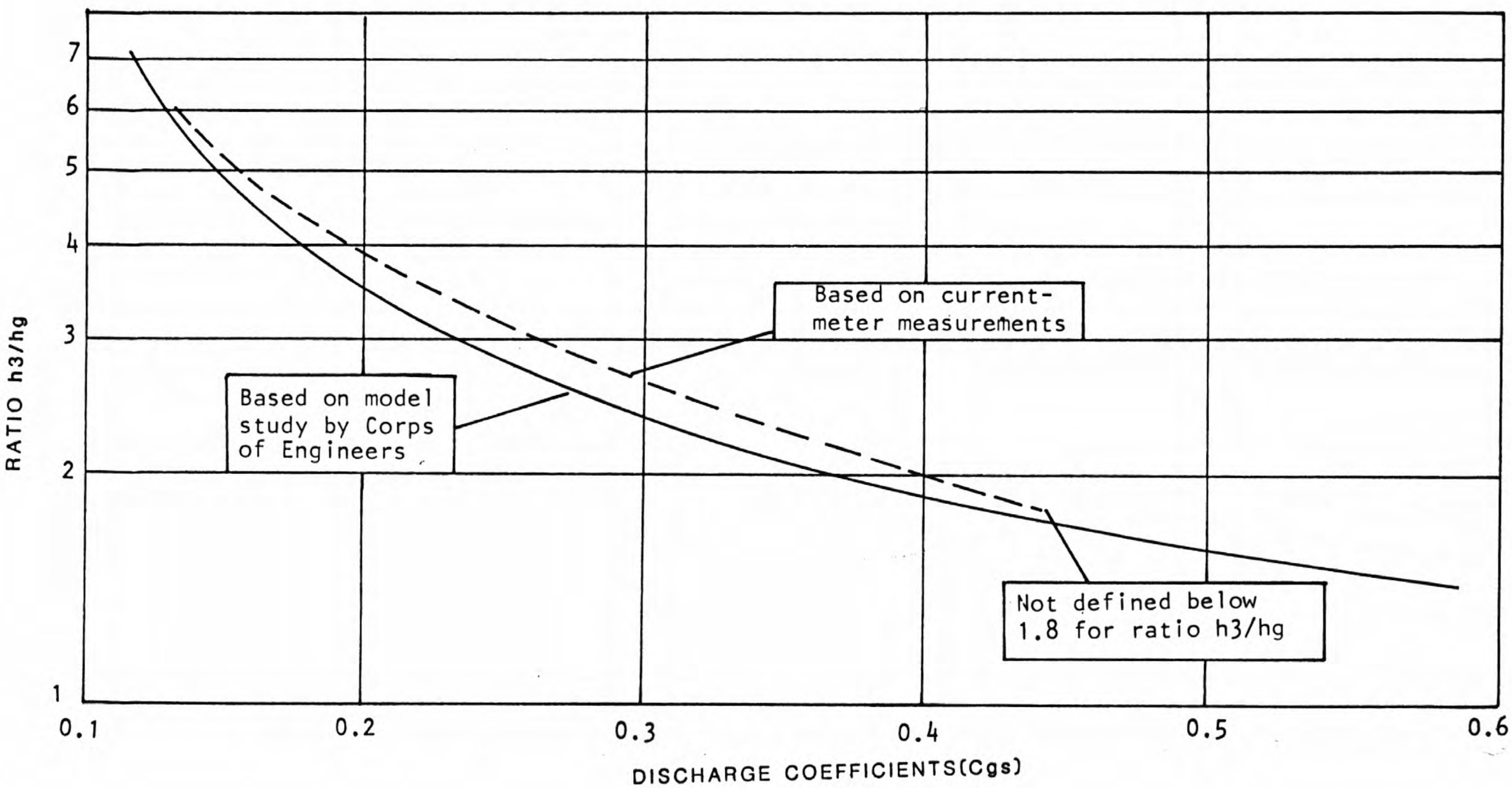


Figure 22.-Discharge coefficient, Cgs, curves for submerged orifice flow at Robert F. Henry Lock and Dam

DISCHARGE COEFFICIENTS

Flow equations are used by the U.S. Army Corps of Engineers, Mobile District, to compute discharge at Robert F. Henry Lock and Dam. Discharge coefficients (C) used in the discharge equations are derived from relations developed in the laboratory. Field verification of the values is needed to determine the accuracy.

Operation of Spillway Gates

The dam spillway has eleven 50-ft wide radial gates situated in individual spillway bays. The gates are operated to maintain a pool stage at 125 ft. When flow is less than 20,000 ft^3/s, all gates are closed and flow passes through the turbines to produce power. Power is generated when the difference between the pool and the tailwater stage is sufficient (about 9 ft). During floods, the gates are raised to maintain a normal pool stage by passing excessive flow. When necessary, the gates are raised above the water surface to allow the flood crest to pass. After the flood crest has past, the gates are lowered into the water sufficiently to maintain a normal pool stage.

Flow at Dam

Flow over the spillway may be classified as orifice or weir, depending on the position of the gates and the relative elevations of headwater and tailwater. Orifice flow occurs when the bottom of the gate is below the water surface, and weir flow occurs when the bottom of the gate is above the water surface. Both flows may be free or submerged depending on the relative elevations of the tailwater and headwater (fig. 23). During normal operations at the dam three types of flow may occur; free orifice, submerged orifice, and submerged weir flow.

Discharge Equations

Discharges are computed through the spillway gates using the following equations (Collins, 1977):

Free orifice flow -

$$Q = C(hg)\ (b)\ [(2g)(h1)]^{0.5} \quad \text{or} \quad Q = C(hg)(b)\sqrt{(2g)(h1)} \qquad (1)$$

Submerged orifice -

$$Q = Cgs(h3)(b)[(2g)(h1-h3)]^{0.5} \quad \text{or} \quad Q = Cgs(h3)(b)\sqrt{(2g)(h1-h3)} \qquad (2)$$

Submerged weir -

$$Q = Cw(Cws)(b)(h1)^{1.5} \quad \text{or} \quad Q = Cw(Cws)(b)\sqrt{(h1)^3} \qquad (3)$$

Where:

b	is	width of gate, in feet
C, Cgs, Cw, Cws	are	coefficients of discharge
g	is	acceleration due to gravity, in feet per second squared
hl	is	difference in pool elevation and spillway crest elevation, in feet
h3	is	difference in tailwater elevation and spillway crest elevation, in feet
hg	is	vertical gage opening above spillway crest, in feet
Q	is	discharge, in cubic feet per second.

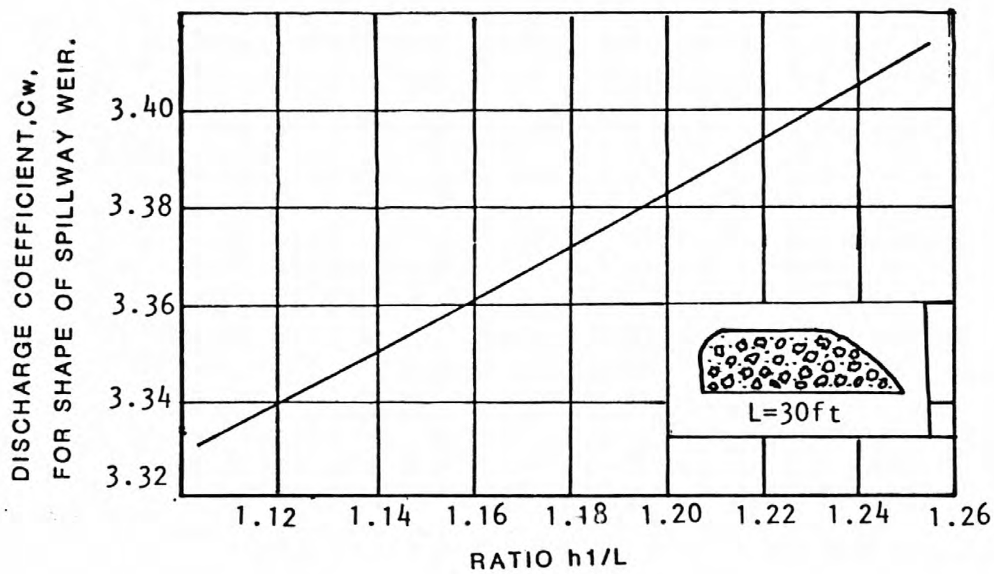


Figure 23.- Discharge coefficient, Cw, curve for submerged weir flow at Robert F. Henry Lock and Dam

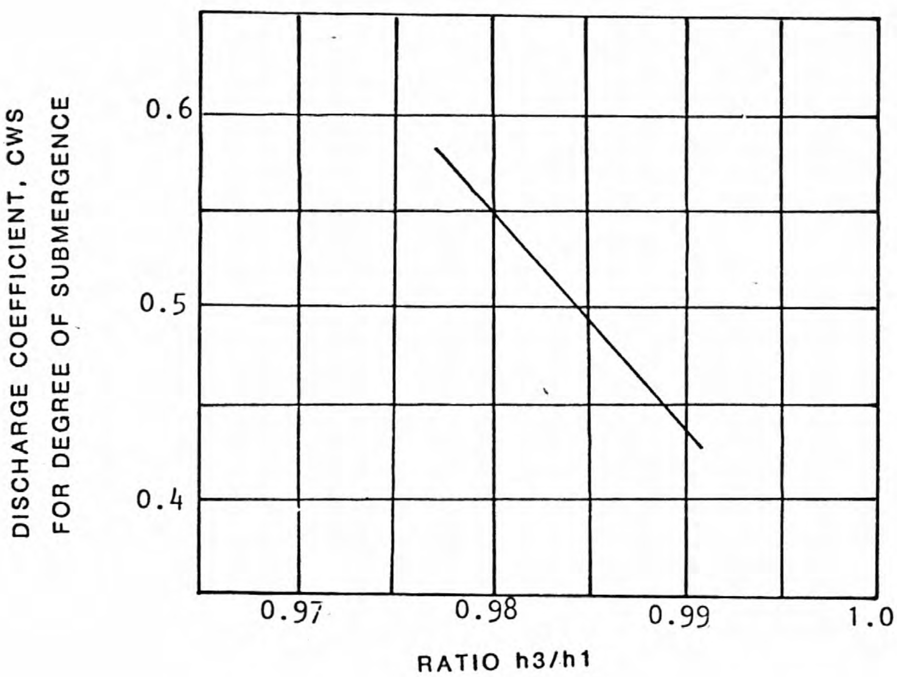


Figure 24.- Discharge coefficient, Cws, Curve for submerged weir flow at Robert F. Henry Lock and Dam

Free orifice flow occurs infrequently at Robert F. Henry Lock and Dam and no discharge measurements were available for this condition. Therefore, no comparisons for C values were made for this equation. Measurements were made in 1976 at each gate forebay for submerged orifice and submerged weir flow with various gate openings. The method of measuring by current-meter in the forebays is described by Collins (1977). Accuracy of this method was verified by occasional simultaneous current-meter measurements made from a boat near the dam. The measured discharges were used in the appropriate equations to compute a value of C. These values were then used to develop average C curves for submerged flow.

Submerged Orifice Flow

The discharge coefficient for submerged orifice flow, Cgs, was computed from the flow equation using data obtained from 39 current-meter measurements. A curve was developed relating the computed values of Cgs to a ratio of head (h3) to gate openings (hg) (fig. 24). The laboratory test curve is also shown in figure 24. Data used to compute the Cgs curve are summarized in table 5.

New gate sills were installed in the spillway in 1977. To determine the new sills effect on the Cgs curve, current-meter measurements were made in February 1981. Results indicate no change in the curve, and additional definition of the lower part of the curve was obtained.

The computed values of Cgs scatter significantly when the ratio h3/hg is less than 1.8 because of the effects of transition from submerged orifice flow to free orifice flow. Therefore, the Cgs curve was not drawn below 1.8 and caution should be exercised when determining Cgs values below this ratio. Discharge measurements are needed when h3/hg is less than about 1.8, to improve or verify the Cgs curve.

Submerged Weir Flow

Two discharge coefficients, Cw and Cws, are required in the discharge equation for the computation of submerged weir flow. The discharge coefficient, Cw, adjusts the equation for the geometric shape of the spillway weir (fig. 25). Values of Cw for this study were taken from a relation for round-crests weirs developed by Hulsing (1968).

The discharge coefficient, Cws, adjusts for the degree of spillway submergence. Values of Cws were computed from the flow equation using discharge from 10 current-meter measurements. The average Cws curve is based on a correlation between Cws values and h3/hl (fig. 26). No laboratory derived values of Cws were available for comparison. Data used to compute the Cws curve are summarized in table 6.

Table 5- Summary of computations for discharge coefficient, Cgs, for submerged orifice flow.

Measurement Number	Date	Elevation in feet		Static head in feet		Height of gate opening in feet	Ratio	Coefficient of discharge, Cgs values		Discharge through gate ft³/s	
		Pool	Tailwater	h_1	h_3	h_g	h_3/h_g	Computed using equation	From average curve	Measured using current-meter	Computed using Cgs from average curve
	1976										
1	3-19	125.10	123.00	34.10	32.00	24.11	1.33	0.806	0.820	15000	15300
2	3-19	125.33	122.35	34.33	31.35	20.82	1.51	0.663	0.640	14400	13900
3	3-20	125.39	120.41	34.39	29.41	16.59	1.77	0.490	0.485	12900	12800
4	3-20	125.33	119.37	34.33	28.37	16.00	1.77	0.450	0.484	12500	13400
5	3-20	125.52	119.63	34.52	28.63	14.87	1.92	0.427	0.428	11900	11900
6	3-20	125.68	119.33	34.68	28.33	14.30	1.98	0.420	0.408	12000	11700
7	3-21	125.69	118.04	34.69	27.04	12.62	2.14	0.373	0.372	11200	11200
8	3-21	125.69	118.04	34.69	27.04	12.62	2.14	0.357	0.372	10700	11200
9	3-23	125.29	116.68	34.29	25.68	12.62	2.04	0.374	0.391	11300	11800
10	3-23	125.55	116.16	34.55	25.16	10.97	2.29	0.346	0.343	10700	10600
11	3-23	125.16	117.21	34.16	26.21	12.62	2.08	0.389	0.381	11500	11300
12	3-24	125.54	114.12	34.54	23.12	9.40	2.47	0.332	0.315	10400	9860
13	3-25	125.50	111.62	34.50	20.62	7.87	2.62	0.308	0.292	9500	9000
14	3-25	125.56	111.00	34.56	20.00	7.38	2.71	0.294	0.284	9000	8760
15	3-25	125.83	110.01	34.83	19.01	4.49	4.23	0.190	0.189	5750	5630
16	3-26	125.50	108.88	34.50	17.88	4.04	4.43	0.178	0.183	5200	5340
17	3-26	125.40	108.66	34.40	17.66	4.04	4.37	0.180	0.185	5230	5360
18	3-27	125.66	110.16	34.66	19.16	4.96	3.66	0.202	0.213	6110	6310
19	3-27	125.60	110.68	34.60	19.68	5.43	3.62	0.215	0.215	6560	6560
20	3-27	125.60	111.53	34.60	20.53	5.91	3.47	0.225	0.224	6940	6930
21	3-28	125.32	114.41	34.32	23.41	7.87	2.98	0.263	0.256	8150	7940
22	3-28	125.38	114.80	34.38	23.80	7.89	3.02	0.267	0.254	8300	7880
23	3-28	125.51	114.86	34.51	23.86	7.89	3.02	0.256	0.254	8000	7930
24	3-29	125.64	114.70	34.64	23.70	6.88	3.44	0.225	0.226	7100	7110
25	4-5	125.03	123.21	34.03	32.21	20.82	1.55	0.632	0.625	11000	10900
	1981										
26	2-11	125.66	106.06	34.66	15.06	4.04	3.73	0.224	0.210	5990	5620
27	2-11	125.67	107.42	34.67	16.42	5.43	3.02	0.253	0.258	7130	7260
28	2-11	125.71	107.65	34.71	16.65	5.43	3.07	0.243	0.253	6910	7180
29	2-11	125.71	107.65	34.71	16.65	3.15	5.29	0.148	0.155	4200	4400
30	2-11	125.64	108.70	34.64	17.70	5.43	3.26	0.241	0.239	7050	6986
31	2-12	125.30	112.44	34.30	21.42	14.30	1.50	0.552	0.550	17000	17000
32	2-12	125.33	112.44	34.33	21.44	14.30	1.50	0.605	0.550	18700	17000
33	2-12	125.36	112.47	34.36	21.47	14.30	1.50	0.625	0.550	19300	17000
34	2-20	125.71	96.70	34.71	5.70	0.99	5.76	0.152	0.143	1870	1760
35	2-20	125.40	97.66	34.41	6.66	5.43	1.23	0.605	0.679	8530	9560
36	2-20	125.31	97.74	34.31	6.74	4.96	1.36	0.549	0.609	7800	8650
37	2-20	125.30	97.68	34.30	6.68	4.49	1.49	0.506	0.553	7130	7790
38	2-20	125.36	97.60	34.36	6.60	4.04	1.63	0.456	0.500	6390	6980
39	2-20	125.36	97.50	34.36	6.50	3.60	1.81	0.423	0.446	5790	6140

Table 6- Summary of computations for discharge coefficient, Cws, for submerged weir flow.

Measurement Number	Date	Elevation in feet		Static head in feet		Ratio	Ratio	Cw	Coefficient of discharge Cws		Discharge through gate (ft^3/s)	
		Pool	Tailwater	h_1	h_3	$h_1/30$	h_3/h_1		Computed using equation	From average curve	Measured	Computed
	1976											
1	3-18	125.12	124.58	34.12	33.58	1.14	0.984	3.36	0.499	0.507	16700	17000
2	3-18	125.03	124.32	34.03	33.32	1.13	0.979	3.35	0.560	0.560	18600	18600
3	4-01	126.09	125.57	35.09	34.57	1.17	0.985	3.37	0.454	0.496	15900	17400
4	4-01	126.64	126.17	35.64	35.17	1.19	0.987	3.38	0.481	0.474	17200	17000
5	4-02	127.20	126.78	36.20	35.78	1.21	0.988	3.39	0.463	0.463	17100	17100
6	4-02	127.39	126.94	36.39	35.94	1.21	0.988	3.39	0.465	0.463	17300	17200
7	4-03	127.18	126.79	36.18	35.79	1.21	0.989	3.39	0.450	0.452	16600	16700
8	4-03	126.92	126.50	35.92	35.50	1.20	0.988	3.39	0.466	0.463	17000	16900
9	4-04	126.02	125.65	35.02	34.65	1.17	0.989	3.37	0.447	0.452	15600	15800
10	4-04	125.98	125.60	34.98	34.60	1.17	0.989	3.37	0.458	0.452	16000	15000

SUMMARY

Data compiled for floods since the late 1800's indicate that the flood of 1886 is the highest known in the vicinity of Montgomery since 1814.

Flood profiles were developed based on gaging-station data and floodmarks for floods in 1948, 1961, 1976, and 1979. From these profiles and results of a step-backwater computer model study, profiles were computed for discharges ranging from 80,000 ft^3/s to 500,000 ft^3/s.

Stage-discharge relations for five sites were developed using data from the step-backwater profiles.

Enveloping curves show the range in stage-discharge relations for gaging stations at Selma and Montgomery.

Discharge-coefficient curves for submerged orifice and submerged weir flow were developed from current-meter measurements and standard discharge equations for flow over the gate-controlled spillways at Robert F. Henry Lock and Dam.

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