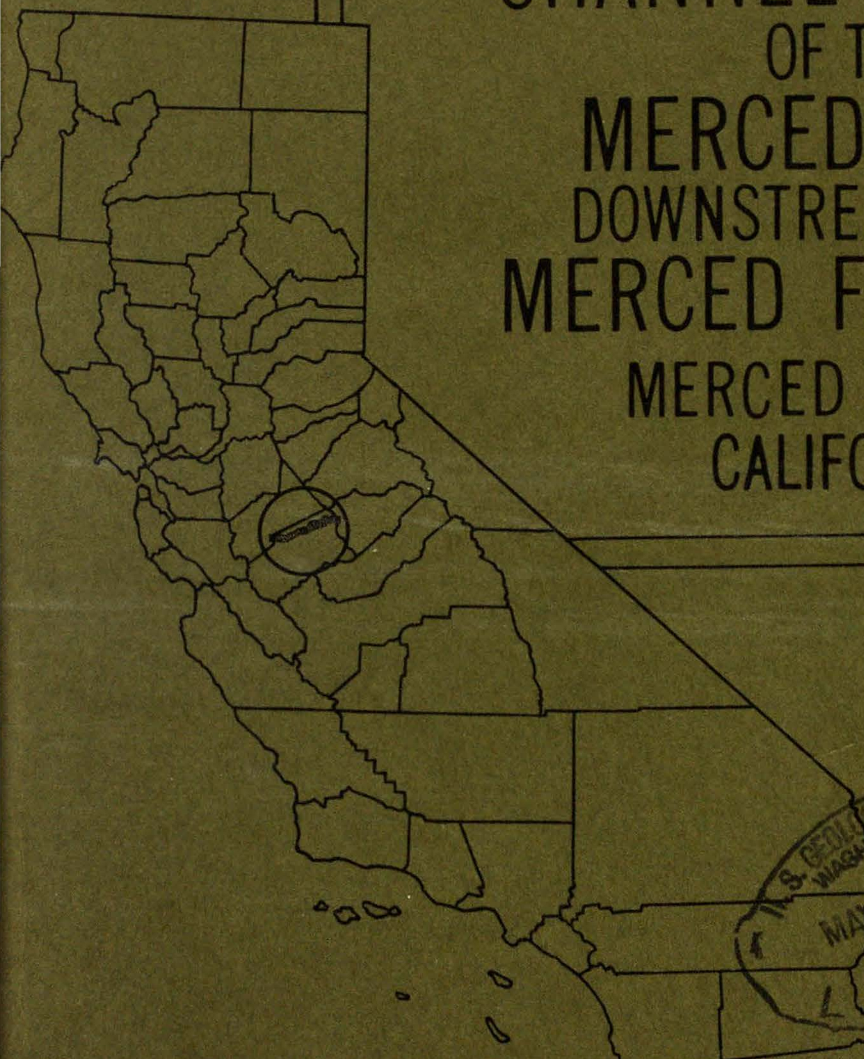
DETERMINATION OF CHANNEL CAPACITY OF THE MERCED RIVER DOWNSTREAM FROM MERCED FALLS DAM MERCED COUNTY CALIFORNIA



Prepared in cooperation with the
CALIFORNIA RECLAMATION BOARD

OPEN-FILE REPORT

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

Menlo Park, California
1968

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J. C. Blodgett, and G. L. Bertoldi , 1938-

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CONTENTS

	Page
Summary and conclusions	1
Introduction	2
Purpose and scope	2
Description of study reach	4
Characteristics of floodflow in the study reach	6
Computation of water-surface profiles	11
Theory	11
Collection of field data	13
Field surveys	13
Roughness coefficients	13
Stage-discharge relations	15
Results of the study	15
Lower subreach	16
Middle subreach	20
Upper subreach	24
References cited	26

ILLUSTRATIONS

[Plates are in pocket]

Plate 1. Map and profile of Merced River, California, showing location of cross sections 1-33.
2. Map and profile of Merced River, California, showing location of cross sections 34-99.
3. Map and profile of Merced River, California, showing location of cross sections 100-141A.
4. Map and profile of Merced River, California, showing location of cross sections 142 and 143.
5. Map and profile of Merced River, California, showing location of cross sections 144-149.

		Page
Figure 1.	Index map	3
2.	Relation of peak discharge and distance traversed between cross section 2 and cross section 149	7
3.	Comparison of water level in Turlock Irrigation District well 314 with discharge of Merced River near Stevinson, gage, for the period 1948-67	10
4.	Block diagram of typical reach	14
5-49.	Cross sections	end of text

Page

Figure 50. Relation between discharge at Merced River near Stevinson, gage and discharge in South, Main, and North Sloughs-- 17
51. Discharge and fall relations at Merced River near Stevinson, gage-- 19
52. Effect of Dry Creek flow on discharge of the Merced River at Cressey-- 25

TABLES

Page

Table 1. Tabulation of June-July 1967 flood discharges at the gaging stations in the study reach---------------- 8
2. Average monthly diversions from the Merced River between Merced Falls and Stevinson gages for the 1967 water year-- 9
3. Elevation at which flows begin in the South, Main, and North Sloughs, and relative flow volumes---------- 16
4. Observed water-surface elevations for selected discharges at the entrance to South, Main, and North Sloughs-- 20
5. Annual maximum peak stages and discharges of the Merced River near Stevinson-------------------------- 21
6. Comparison of measured and computed stage-discharge relations-- 23

APPENDIXES

Page

A. Elevation of water surface for minimum measured backwater conditions at cross sections between 26,317 feet and 171,200 feet upstream from the gaging station on the San Joaquin River near Newman-- 27
B. Elevation of water surface for maximum measured backwater conditions at cross sections between 26,317 feet and 42,660 feet upstream from the gaging station on the San Joaquin River near Newman-- 29

DETERMINATION OF CHANNEL CAPACITY OF THE MERCED RIVER DOWNSTREAM FROM MERCED FALLS DAM, MERCED COUNTY, CALIFORNIA

By J. C. Blodgett and G. L. Bertoldi

SUMMARY AND CONCLUSIONS

This study evaluates the adequacy of a reach of the Merced River between Merced Falls and the confluence with the San Joaquin River to carry flood releases from New Exchequer and McSwain Dams and Reservoirs. The flood release from these reservoirs is to be restricted so that flows will not exceed 6,000 cfs (cubic feet per second) in the Merced River at the gaging station near Stevinson (about 5 miles upstream from the mouth). Computed floodwater profiles based on channel conditions in late 1967 and observed water-surface profiles for historic floods were used in the analysis. The conclusions reached are contingent on there being no levee failures during periods of high flow.

Evaluation of historical flood records at gaging stations between Merced Falls and Stevinson indicates that a reduction in peak discharge occurs as the floods traverse the study reach. Reduction in peak discharge is dependent on river stage, prior river flows, magnitude of tributary inflow, bank storage and infiltration to the ground-water reservoir, and diversion for irrigation. For example, in late June 1967 a peak discharge of 9,860 cfs at Merced Falls was reduced by irrigation diversions to 6,850 cfs at a site 3 miles downstream and further reduced to 6,490 cfs at the gaging station near Stevinson.

Backwater effect from high stages on the San Joaquin River will increase stages on the Merced River and the effect may extend as far as 8 miles upstream from the junction of the two rivers. Backwater may cause some overbank flooding along the 8-mile reach when Merced River flows at Stevinson exceed 4,000 cfs. Most of the area subject to such inundation lies between levees that are a considerable distance--up to half a mile--from the main channel. In the reach upstream from the backwater effect, a discharge of 4,500 cfs near Stevinson will cause flooding at two locations, one of which is about 10 miles upstream and the other about 18.5 miles upstream from the confluence. When the discharge is 6,000 cfs, additional sites will be inundated, particularly the 20.5-mile reach of channel downstream from the Highway 99 crossing.

A discharge of 6,000 cfs will pass through all bridge openings in the reach but overbank flooding may occur if debris should lodge against the bridge piers and significantly reduce the waterway area.

Peak flows in Dry Creek, as measured 18.7 miles upstream from the mouth of Dry Creek, will be attenuated due to channel storage and will increase the discharge of the Merced River at Cressey by only about 50 percent of the Dry Creek peak discharge. Furthermore, Dry Creek seldom carries floodflows during periods of high water on the Merced River.

INTRODUCTION

Purpose and Scope

At the request of the Reclamation Board, State of California, the U.S. Geological Survey made a study of the channel capacity of a 55-mile reach of the Merced River in Merced County, California (fig. 1, pls. 1-5). The study reach commences at the confluence of the Merced and San Joaquin Rivers and extends upstream to the gaging station on the Merced River below Merced Falls Dam. A gaging station on the San Joaquin River near Newman, 0.14 mile downstream from the mouth, was used to obtain river stages at the confluence. Therefore, river distances given in this report are referenced to that gage.

This study was made to determine the water-surface profiles that would result from selected flood releases from New Exchequer and McSwain Dams and Reservoirs, located about 6 miles upstream from the upper end of the study reach. The discharges studied ranged from 4,000 to 9,000 cfs. A check on the computations was provided by utilizing streamflow data obtained during the flood of June-July 1967.

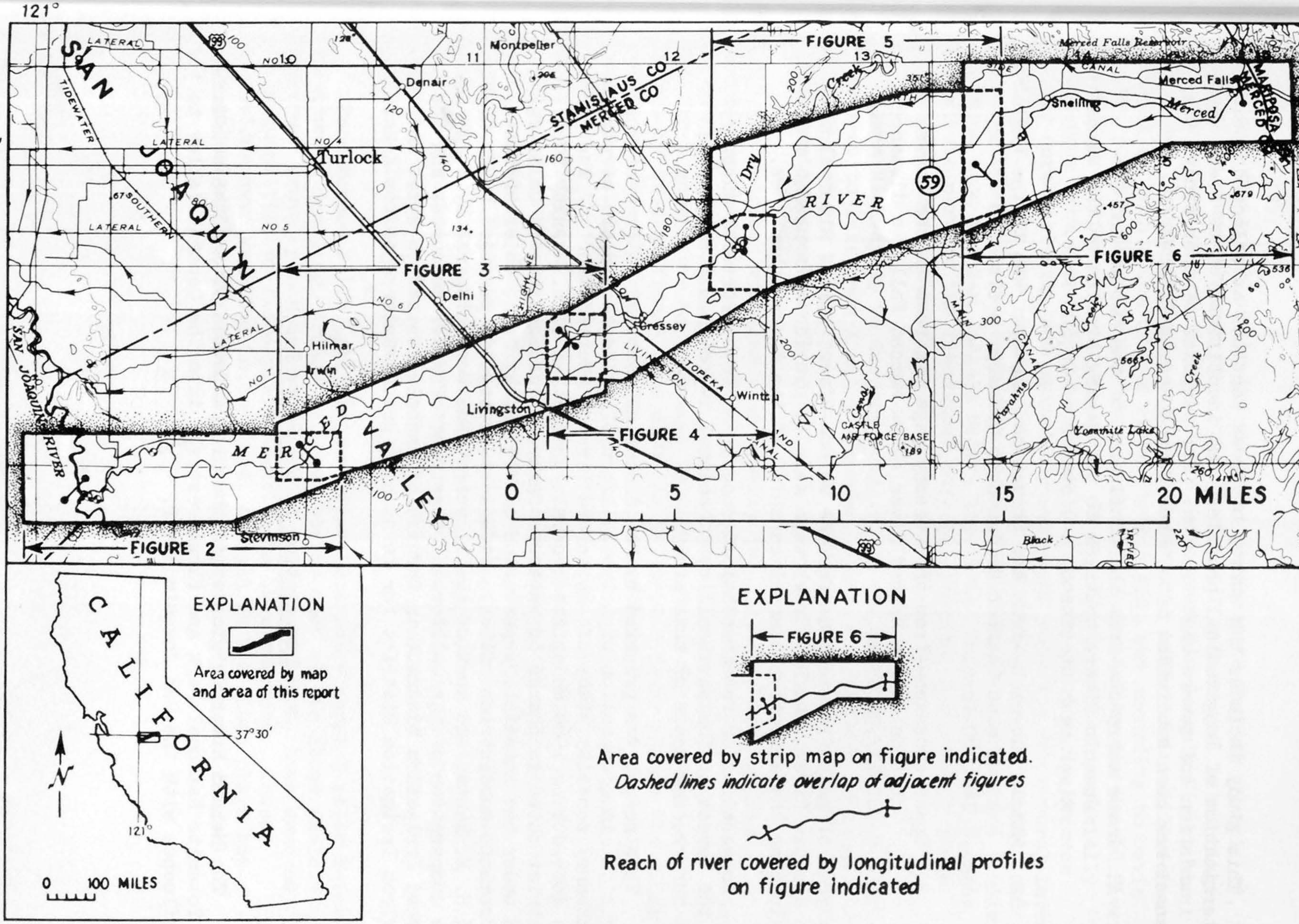


Figure 1.—Index map showing location of study area and strip maps used in report.

This study includes the survey of cross sections at selected locations, determination of longitudinal profiles, and identification of areas subject to inundation and severe bank erosion. For purposes of this study, the channel has been subdivided into three subreaches as follows:

1. Lower subreach--from the confluence of the Merced and San Joaquin Rivers upstream to the gaging station on the Merced River near Stevinson, a distance of 25,600 feet.

2. Middle subreach--from the Stevinson gage upstream to the gaging station at Shaffer Bridge near Cressey, a distance of 144,880 feet.

3. Upper subreach--from the Cressey gage upstream to the gaging station on the Merced River below Merced Falls, a distance of 117,340 feet.

Analyses of peak-flow records for the Merced River, Merced River Slough, San Joaquin River, and Dry Creek were made to provide data needed for evaluating the effective channel capacity of the three subreaches.

Suggestions or recommendations concerning the structural adequacy of the levees or the advisability of channel improvements are beyond the purpose and scope of this study.

This report was prepared by the Geological Survey, Water Resources Division, in cooperation with the California Reclamation Board as part of a channel capacity study of the Merced River in Merced County. The work was done during 1968 under the general supervision of R. Stanley Lord, district chief in charge of water-resources investigations in California, and under the immediate supervision of Willard W. Dean, chief of the Sacramento subdistrict office. Technical review was provided by H. A. Ray and S. E. Rantz, and most of the illustrations were prepared by M. E. Royce. The cooperation of the California Department of Water Resources and the Merced Irrigation District in furnishing certain streamflow data, and the Turlock Irrigation District for furnishing ground-water data, is acknowledged.

Description of the Study Reach

The Merced River originates in the Sierra Nevada within the boundaries of Yosemite National Park and flows westerly into the Central Valley to the confluence with the San Joaquin River.

From the mouth of the Merced River upstream to Highway 99 (pl. 1), the channel is characterized by a wide flood plain and several meandering sloughs. Near the confluence of the Merced and San Joaquin Rivers the river and slough gradients are about 2 feet per mile. Natural levees are present on both sides of the river flood plain, and according to Davis (1959, p. 24), these levees were apparently deposited by the Merced River preceding the present entrenchment. In the lower reaches, especially downstream from the Millikin Bridge (pl. 1), the flood plain becomes nearly unrecognizable and manmade levees supplement the low natural levees and banks. Here the channel bed, banks, and levees are very sandy, and scouring occurs during each high water. At sites between 0.4 and 4 miles upstream from the San Joaquin River, the South, Main, and North Sloughs distribute part of the floodflows directly into the San Joaquin River downstream from the confluence of the main river channel.

From Highway 99 to Cressey (pls. 2, 3) the width of the flood plain decreases to less than 1 mile. However, the river gradient increases to about 3 feet per mile, the channel is deep and well defined, and consequently, overbank flooding in this subreach is minimal.

The subreach upstream from Cressey (pl. 3), is characterized by a meandering channel with associated sloughs and a flood plain that widens to as much as 3 miles. Just downstream from the Shaffer Bridge (pl. 3), Dry Creek enters the Merced River from the north. This is the only tributary of significant size entering the Merced River within the study reach. At one time the main channel of the Merced River apparently occupied what is now known as Ingalsbe Slough. This slough bypasses Snelling on the north and re-enters the Merced River 0.8 mile upstream from Shaffer Bridge (pls. 3, 4). During periods of high water, water may be diverted from the main channel through Hopeton and Dana Sloughs about 3 miles upstream from the bridge on State Highway 59. Flows from these sloughs enter Ingalsbe Slough about 1 mile upstream from the confluence of Ingalsbe Slough and the Merced River. The gradient of the main channel upstream from Cressey averages 7 feet per mile, 2 times greater than that below Cressey. Because the channel gradient is steep, overbank flooding is minimal.

From Merced Falls (pl. 5) downstream to a point about 3 miles below the town of Snelling, dredge tailings occupy much of the flood plain. Numerous gravel-plant operations, both past and present, have removed large quantities of flood-plain alluvium in this subreach. These operations have resulted in continual changes in the flood plain but have not significantly changed the carrying capacity of the channel.

Characteristics of Floodflow in the Study Reach

As floodflows traverse the Merced River between Merced Falls and the San Joaquin River, irrigation diversion, channel storage, and losses to the ground-water reservoir tend to reduce the peak flow. Reduction in peak discharge is related to the magnitude of peak discharge and the time of the year in which the flood occurs, as shown in figure 2. For the extreme cases shown in figure 2, peak discharges of the floods of December 1950 and December 3, 1965 were reduced 71 percent and 31 percent, respectively, after the peaks had traversed the reach.

Channel storage is a major factor in reducing peak stages as the flood traverses the reach, unless large sustained releases from upstream reservoirs or floods have occurred which still occupy a large part of the channel. These earlier peaks, which need not be large enough to cause overbank flow, tend to fill the available channel storage. The influence of antecedent channel conditions on peak flows is illustrated by the flood of June-July 1967 (table 1) which actually consisted of three peaks occurring over a 4-day period at the Merced Falls gage and over a 5-day period at the Stevinson gage. During the June-July flood, channel storage was effective in reducing the June 28 peak at Merced Falls from the highest of the three to the lowest by the time the crest had reached the gaging station on the Merced River below Snelling (pl. 5). Between the Snelling and Stevinson gages channel storage became less effective as the flood period progressed, as indicated by the reduced change in flood discharge between the gages for the later peaks.

The reduction in peak flow caused by diversions is significant only during the irrigation season as can be seen in table 2.

In addition to irrigation diversions and channel storage, floodflows may be reduced by infiltration and bank storage. Part of the change in peak discharges between gaging stations noted during the flood of June-July 1967 (table 1) is probably due to bank storage and infiltration to the ground-water reservoir. As with channel storage, the influence of bank storage and infiltration losses are affected primarily by antecedent runoff conditions and to a lesser degree by precipitation and evapotranspiration.

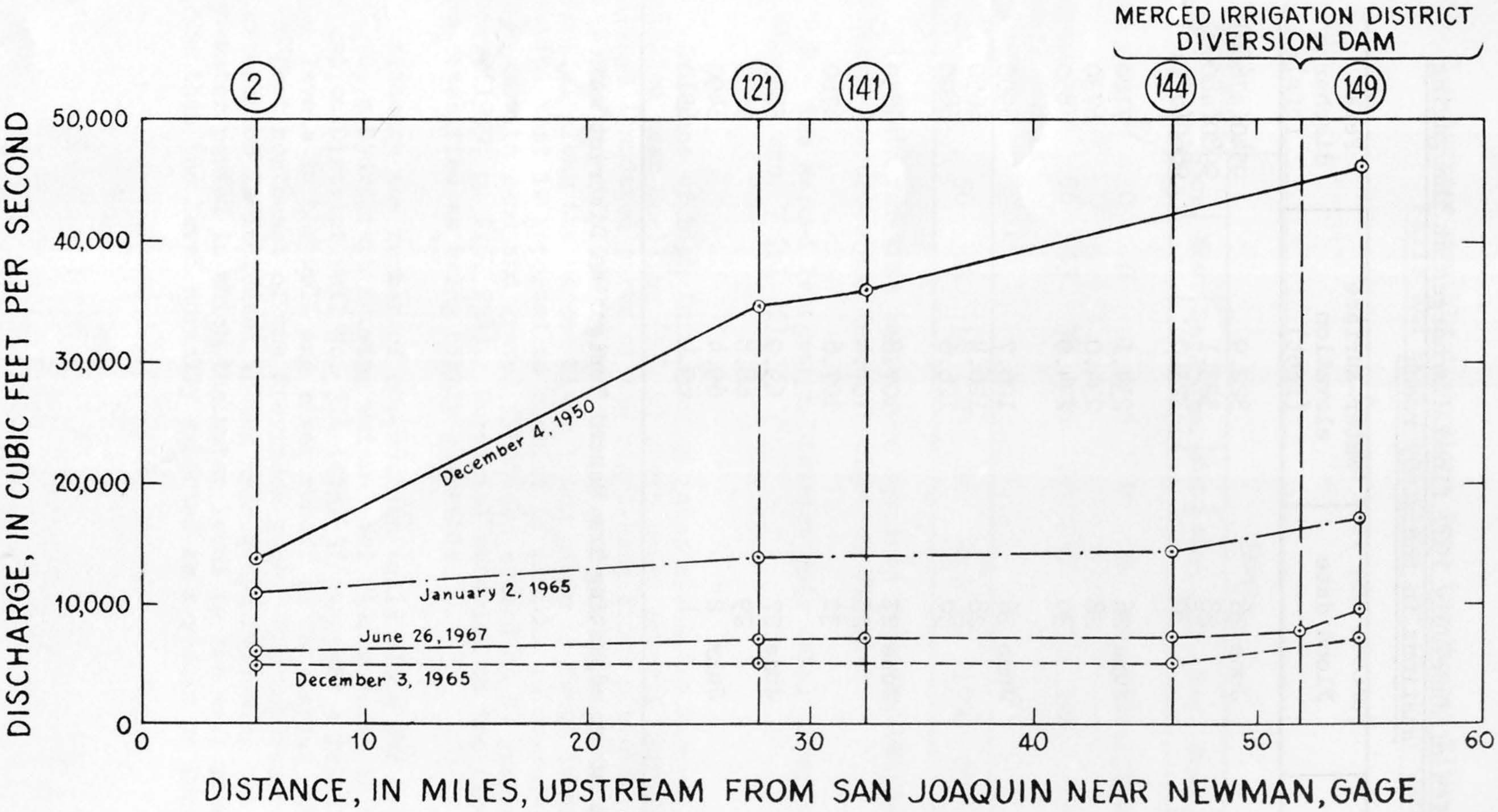


Figure 2.—Relation of peak discharge and distance traversed between cross section (2) and cross section (149).

Table 1.--Tabulation of June-July 1967 flood discharges at the gaging stations in the study reach

Gaging station	Flood date	Water-surface elevation (feet)	Peak discharge (cfs)
Merced River near Merced Falls	June 26	322.9	9540(a7660)
	28	323.1	9980(a8060)
	30	323.0	9860(a7910)
Merced River below Snelling (DWR gage)	June 26	234.1	7100
	28	234.0	6730
	30	234.0	6850
Merced River at Shaffer Bridge near Cressey	June 26	126.7	6900
	28	126.4	6650
	30	126.6	6800
Merced River at Cressey (DWR gage)	June 27	107.8	6850
	28	107.4	6510
	30	107.6	6780
Merced River near Stevinson	June 27	69.0	6080
	29	68.8	5880
	July 2	69.4	6490
	4	69.4	6510

a. Discharge after adjusting for Merced Irrigation District main canal diversion.

Table 2.--Average monthly diversions from the Merced River between Merced Falls and Stevinson gages for the 1967 water year

(cubic feet per second)

Diversions[1]	Month											
	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Main canal[2]	0	0	0	0	0	0	320	1400	1750	1950	1830	1440
Pumpage and small canals	56	53	38	25	26	47	73	190	254	255	230	171
Total	56	53	38	25	26	47	393	1590	2004	2205	2060	1611

1. Data for this table were obtained from the California Department of Water Resources and Merced Irrigation District.

2. The Merced Irrigation District main canal diversion is located in the SW$\frac{1}{4}$ sec. 7 (pl. 5).

Evidence of high infiltration rates along the Merced River was indicated near Snelling (pl. 5) where water seepage through the dredge tailings inundated parts of the right-bank flood plain during the high water of June-July 1967. Additional evidence of high infiltration potential along the Merced River flood plain is found in the underlying material that is reported to consist of medium- to coarse-grained alluvium with sand and gravel fractions exceeding 50 percent (Davis and others, 1959, p. 215, 254). Surficial material on the flood plain is also classified as being highly permeable.

Although the number of observation wells along the Merced River is limited, a hydrograph showing water-level fluctuations in Turlock Irrigation District well No. 314 (fig. 3) suggests a relation between water levels in the well and river runoff in this area. The well is 2,700 feet northeast of the Stevinson gage. The importance of this interrelation is indicated in the hydrograph following the June-July 1967 high-water period in which the water level in the well rose about 2 feet higher than the level normally expected as a result of irrigation recharge.

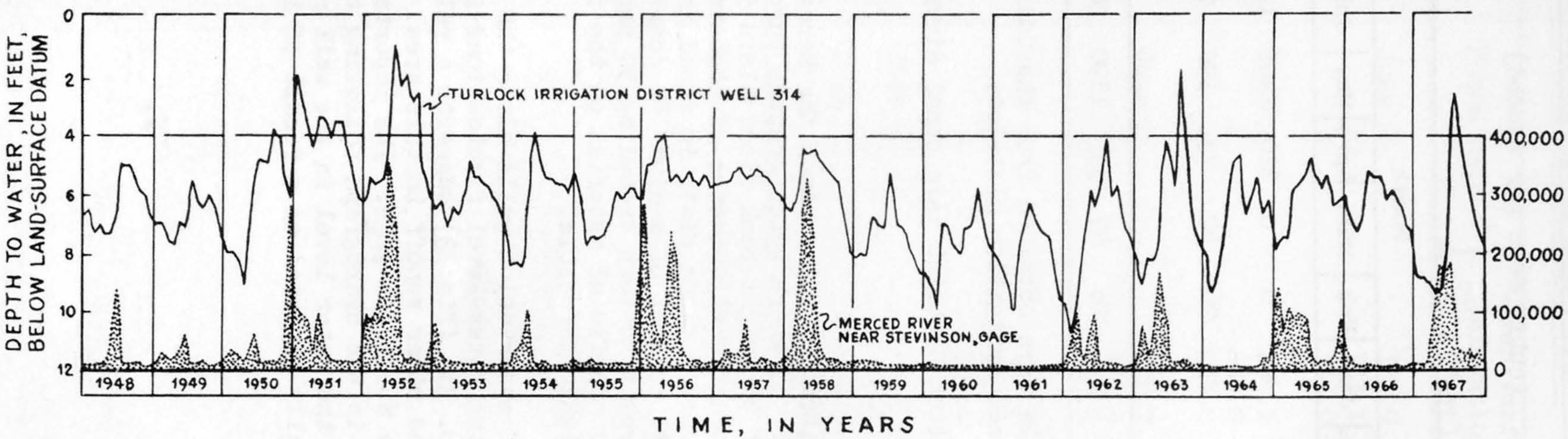


Figure 3.—Comparison of water level in Turlock Irrigation District well 314 with discharge of Merced River near Stevinson, gage, for the period 1948-1967.

COMPUTATION OF WATER-SURFACE PROFILES

The step-backwater method is used to determine water-surface profiles in natural channels for given discharges. Data necessary for the computations include a survey to determine the geometry of the stream channel, selection of channel-roughness coefficients, and a measured or theoretical stage-discharge relation at the downstream end of the reach. The water-surface profile corresponding to any known or assumed discharge may be computed from this information.

Theory

The underlying theory for the computation of water-surface elevations, as discussed by Bailey and Ray (1966), is the principle of the conservation of energy between two cross sections of a stream. The basic equation is expressed as follows:

$$h_d + h_{v_d} + h_f + h_e = h_u + h_{v_u}$$

where subscripts d and u refer to the downstream and upstream cross sections, respectively; h is elevation of the water surface, in feet, above a datum plane; h_v is velocity head, in feet, at a cross section; h_f is the friction loss, in feet, between cross sections; and h_e is the energy loss, in feet, resulting from deceleration of flow in an expanding subreach between cross sections. The individual terms in the basic equation are computed as follows:

$$h_v = V^2/2g \qquad (1)$$

where V is average velocity, in feet per second, in a cross section and g is the acceleration of gravity (32.2 ft/sec^2).

$$h_f = \frac{LQ^2}{K_d K_u} \qquad (2)$$

where L is the distance, in feet, between cross sections; Q is discharge, in cubic feet per second; and K is the conveyance, at a cross section. The equation to compute K is

$$K = \frac{1.486AR^{2/3}}{n} \qquad (3)$$

where A is the area, in square feet, of a cross section; R is the hydraulic radius, in feet, of a cross section; and n is the Manning roughness coefficient.

$$h_e = k(\Delta h_v) \tag{4}$$

where k = 0 for a contracting subreach between cross sections and k = 0.5 for an expanding subreach and $\Delta h_v = h_{v_u} - h_{v_d}$.

The step-backwater analysis consists of solving the basic equation by trial-and-error computations within specified tolerances. This method is one of several used in the computation of gradually varied flow profiles (Chow, 1959, chap. 10). It is applicable to subcritical or supercritical flow provided that subcritical flow computations progress in the upstream direction and supercritical flow computations progress downstream. All flows in this study were subcritical. The theory underlying the basic equations assumes that uniform-flow formulas are applicable to gradually varied flow conditions. The following conditions are assumed to be in effect:

1. Flow is steady between cross sections.
2. Slope is small so that depths perpendicular to the water surface can be considered equal to vertical depths.
3. Water-surface elevation is level across a cross section.
4. Effects of sediment and air entrainment are negligible.
5. All energy losses are included in the h_f and h_e terms.

The step-backwater method is generally regarded as best for the computation of flow profiles in natural channels (Chow, 1959, p. 267). This method has two principal advantages: (1) The maximum possible use of channel geometry is permitted, and (2) knowledge of the water-surface elevation is not a prerequisite. This is because several water-surface profiles of the same discharge starting with different water-surface elevations at the initial section will tend to converge to a single profile if the reach is of adequate length. Where these profiles converge, the computed elevation will be theoretically correct.

Results of the step-backwater computations for the Merced River are given in appendixes A and B of this report. The water-surface elevations given in the appendixes were derived assuming that levees would be extended high enough to contain the maximum discharge indicated.

Collection of Field Data

Field Surveys

The pertinent geometry of the stream channel was determined by transit-stadia survey. Vertical control was established at several locations throughout the study reach from U.S. Coast and Geodetic Survey bench marks. All elevations given in this report are referenced to mean sea-level datum (1929 datum, 1965-66 adjustment).

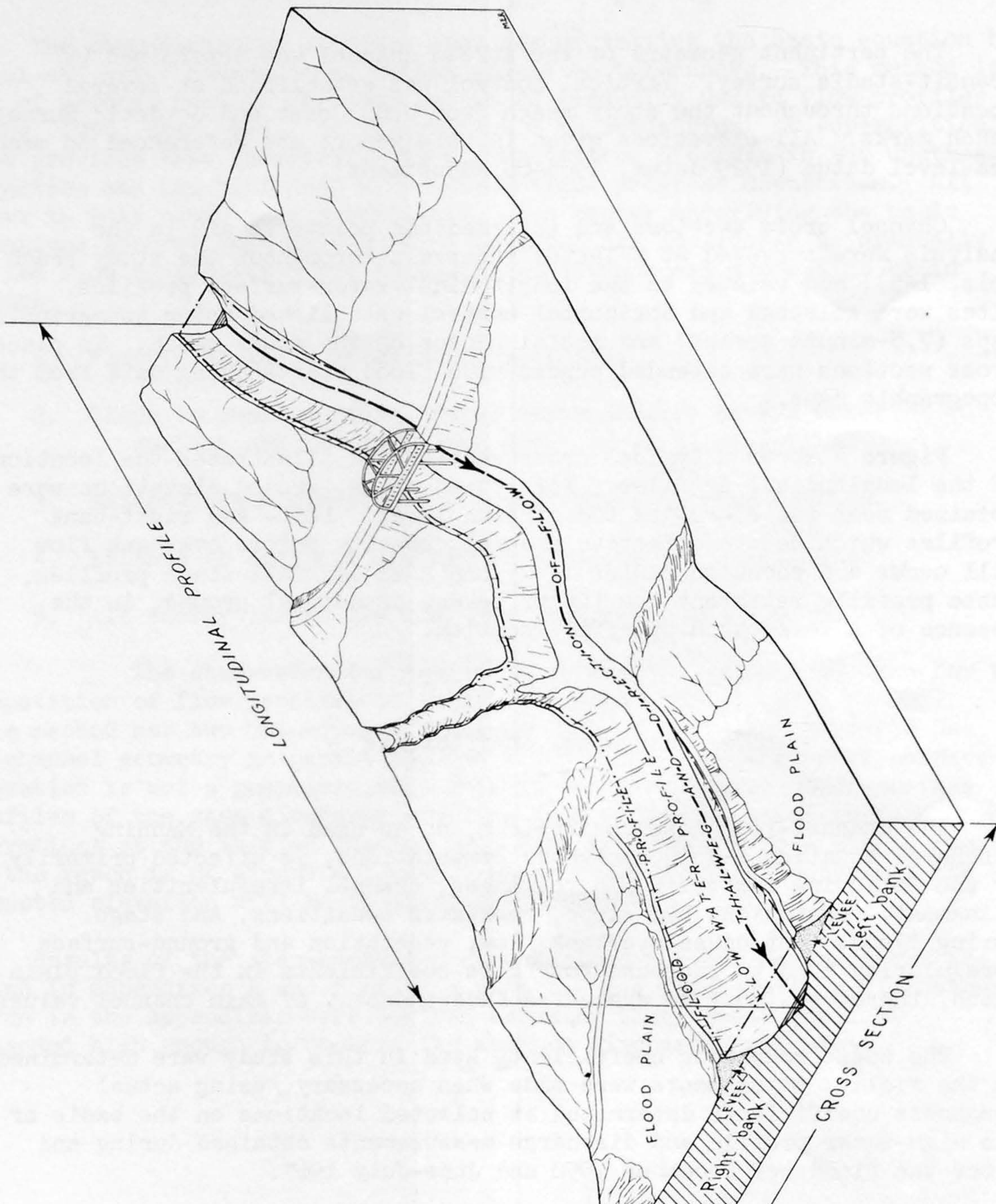
Channel cross sections and intermediate points to aid in the analysis were surveyed at selected intervals throughout the study reach (pls. 1-5), and related to the longitudinal water-surface profiles. Sites were selected and horizontal control established using topographic maps (7.5-minute series) and aerial photos of the study reach. In places, cross sections were extended across wide flood plains using data from the topographic maps.

Figure 4 shows a typical cross section and illustrates the location of the longitudinal profiles. For leveed areas, ground elevations were obtained near the shoreward toe of each levee. Left- and right-bank profiles which denote effective channel capacity before overbank flow will occur are shown on plates 1, 2, and 3 as bankfull stage profiles. These profiles represent the top of levee, or natural ground, in the absence of a levee at a specific location.

Roughness Coefficients

The channel-roughness coefficient, n, as used in the Manning discharge equation and in backwater computations, is affected primarily by the following factors: Bed roughness, channel irregularities and alinement, vegetation, bed slope, backwater conditions, and stage. During floods that cause overbank flow, vegetation and ground-surface irregularity tend to increase roughness coefficients in the flood plain which, therefore, must be evaluated independently of main channel values.

The basic roughness coefficients used in this study were determined in the field. Adjustments were made when necessary, using actual roughness coefficients determined at selected locations on the basis of the high-water profile and discharge measurements obtained during and after the floods of December 1950 and June-July 1967.


LONGITUDINAL PROFILE
DIRECTION OF FLOW
FLOOD PROFILE
LOW-WATER PROFILE AND THALWEG
FLOOD PLAIN
FLOOD PLAIN
LEVEE
Left bank
LEVEE
Right bank
CROSS SECTION

Stage-Discharge Relations

The starting point for backwater computations is a downstream site where the stage-discharge relation is known or can be derived. For the middle and upper subreaches, (pls. 2-5), the stage-discharge relation used was developed from data obtained at the gaging station on the Merced River near Stevinson for conditions of minimum measured backwater from the San Joaquin River. This rating was prepared using discharge measurements made at flows ranging from 42 to 10,500 cfs. Water-surface elevations corresponding to selected discharges were used to progress upstream by the step-backwater technique. The effect of backwater on stages in the lower subreach, downstream from the gage near Stevinson, is discussed later in this report under the heading Lower Subreach.

Stage-discharge relations defined by current-meter measurements were also available for the gaging station on the Merced River at Cressey (operated by the California Department of Water Resources) and at Shaffer Bridge near Cressey. The stage-discharge relations at these sites were used to compare computed water-surface elevations with the observed water-surface elevations.

RESULTS OF THE STUDY

The carrying capacity of the Merced River has been altered considerably by the addition of earthfill levees (figs. 5-49) which provide the height needed to contain floodflows. Appraisal of the structural adequacy of the levee system is beyond the scope of this study. However, a considerable area of river bottom land is being reclaimed for agricultural use and protected by earthfill levees with resultant reduction in the channel size to handle floods. The analyses that follow, therefore are contingent on there being (1) no levee failures, (2) no increase in river bottom encroachment by land reclamation and levee construction, (3) no reduction in channel carrying capacity due to channel encroachment by trees and brush which would normally be scoured out of the main channel during floods, and (4) no decrease in the carrying capacity of the sloughs as a result of debris and fill dumped into the sloughs by the stream or by land-clearing operations.

The design flood of 6,000 cfs will pass through all bridge openings without complications except that some overbank flooding may result if floating debris should lodge against the Millikin and Snelling Road bridges and significantly reduce the waterway area.

Lower Subreach

The lower subreach extending from the mouth of the Merced River to the Stevinson gage is characterized by a meadering main channel with a wide flood plain. The carrying capacity of the main channel has been increased by the addition of earthfill levees which provide the height needed to contain the floodflows in this subreach. Some of these levees are located as much as half a mile from the main channel as shown on plate 1. As a result, much of the land adjacent to the river is subject to inundation whenever flows exceed about 4,000 cfs at the Stevinson gage. The areas inundated by a flood discharge of 6,500 cfs are shown on plates 1-5. Below the Stevinson gage, the South, Main, and North Sloughs leave the Merced River to the north (pl. 1) and discharge directly into the San Joaquin River, providing relief from overbank flooding. The stages at which flow will occur and the relative flow volumes in each slough are shown in table 3. The discharge in each slough and the total slough discharge will vary depending on the stage of the San Joaquin River and the discharge in the Merced River just upstream from the sloughs. The backwater effect of the San Joaquin River on flow in the Merced River and its sloughs increases with increase in stage of the San Joaquin River and decreases with increase in discharge in the Merced River. San Joaquin River stages are recorded continuously at a gage near Newman, 0.14 mile downstream from the mouth of the Merced River.

Table 3.--Elevation at which flows begin in the South, Main, and North Sloughs, and relative flow volumes

Slough	Elevation at head of slough where flow begins (feet above mean sea level)	Proportion of total slough flow, in percent		
		Mean	Minimum recorded	Maximum recorded
South	59.7	26	0	69
Main	61.9	69	31	100
North	67.2	5	0	27

The importance of these sloughs in conveying floodwater, and the backwater effect from high San Joaquin River stages on the discharge in the sloughs, is illustrated by the enveloping lines shown in figure 50. Results given in table 3 and figure 50 were derived using flood data obtained since 1950. At a discharge of 6,000 cfs, between 3 and 63 percent of the flow of the Merced River may enter the San Joaquin River through these sloughs, depending on the stage of the San Joaquin River. The South Slough is located on the right bank of the San Joaquin flood plain and flow in this slough may occur whenever the stage of the San Joaquin River exceeds about 60 feet and inundates the flood plain. Thus, a portion of the discharge indicated for total slough flow in figure 50 may include flow from both the San Joaquin and Merced Rivers.

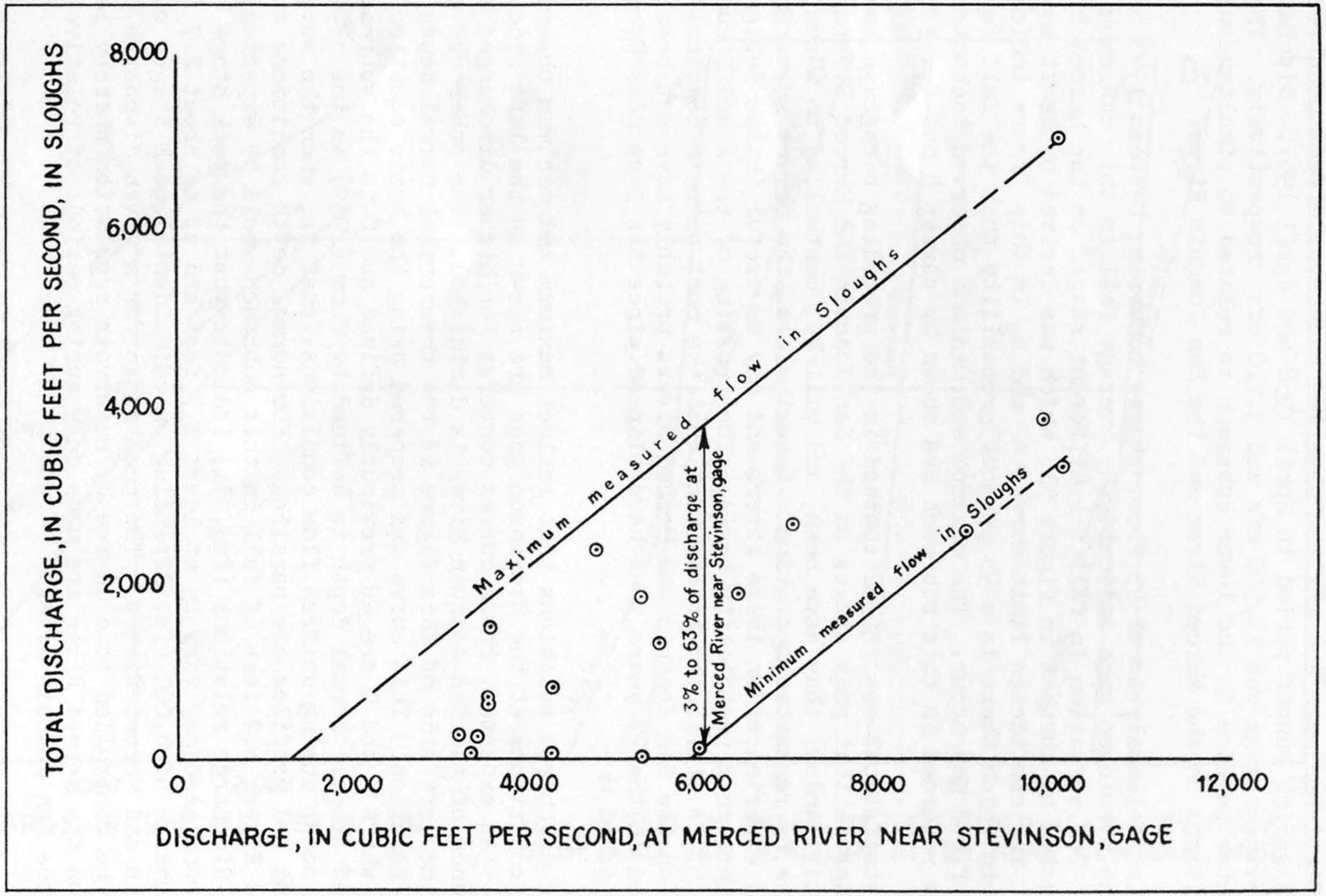


Figure 50.—Relation between discharge at Merced River near Stevinson, gage and discharge in South, Main, and North Sloughs.

Backwater conditions caused by the San Joaquin River extend upstream past the Stevinson gage and maximum observed backwater effect occurred during the spring runoff period in April 1958 and April 1967. Discharge at the Stevinson gage was 11,500 cfs and 3,710 cfs, respectively. The fall in water surface in the lower subreach is related to simultaneous flow conditions on the Merced River and the San Joaquin River.

Statistical analysis of 95 flood stages occurring between 1945 and 1967 at the Stevinson gage indicate the average fall in this subreach is 8.3 feet. The variation in fall for different stages on the Merced River is illustrated by curve A in figure 51, which was derived by least squares analysis. The confidence limit curves A_1 and A_2 in this figure indicate the range in which there is a 95 percent probability that the fall for any given flood may occur. The maximum and minimum observed backwater conditions recorded in this subreach are shown by curves B and B_1.

The statistical analysis discussed in the preceding paragraph has some shortcomings in that peak stages on the San Joaquin and Merced Rivers are not strictly random; they have been, and will be, controlled to a considerable degree by reservoir operation. Nevertheless, the curves summarize past flood experiences on these rivers and may be useful in the design of levee heights on a probability basis. The crossing of the A and B curves in figure 51 is attributed to the fact that the most extreme combinations of stages on the San Joaquin and Merced Rivers probably have not been experienced in the 27 years that have elapsed since the gage near Stevinson was established.

Stage-discharge relations that reflect maximum and minimum observed backwater conditions at the Stevinson gage are shown on the left side of figure 51. As expected, these curves converge for higher discharges since the influence of the San Joaquin River is diminished. The other curve shown on the left side of this figure is the theoretical normal depth stage-discharge relation. This curve was prepared using the basic equation $K = Q/S^{-\frac{1}{2}}$ where K and Q are as previously defined and S is the subreach channel bed slope. Normal depth is defined by Chow (1959) as the depth that will occur during uniform flow conditions--that is, when the water surface and bed profiles are parallel. For normal depth conditions on the Merced River, 6.2 feet of fall in this subreach would be expected. The stage-discharge relations (fig. 51) indicate that the peak stage at the Stevinson gage may vary by at least 1.7 feet and up to about 2.7 feet for a discharge of 6,000 cfs, depending on coincidental peak stages on the San Joaquin and Merced Rivers. The peak stage for a given discharge can therefore be controlled to a degree by reservoir regulation whereby large releases on the Merced River are made only during periods of relatively low stage on the San Joaquin River.

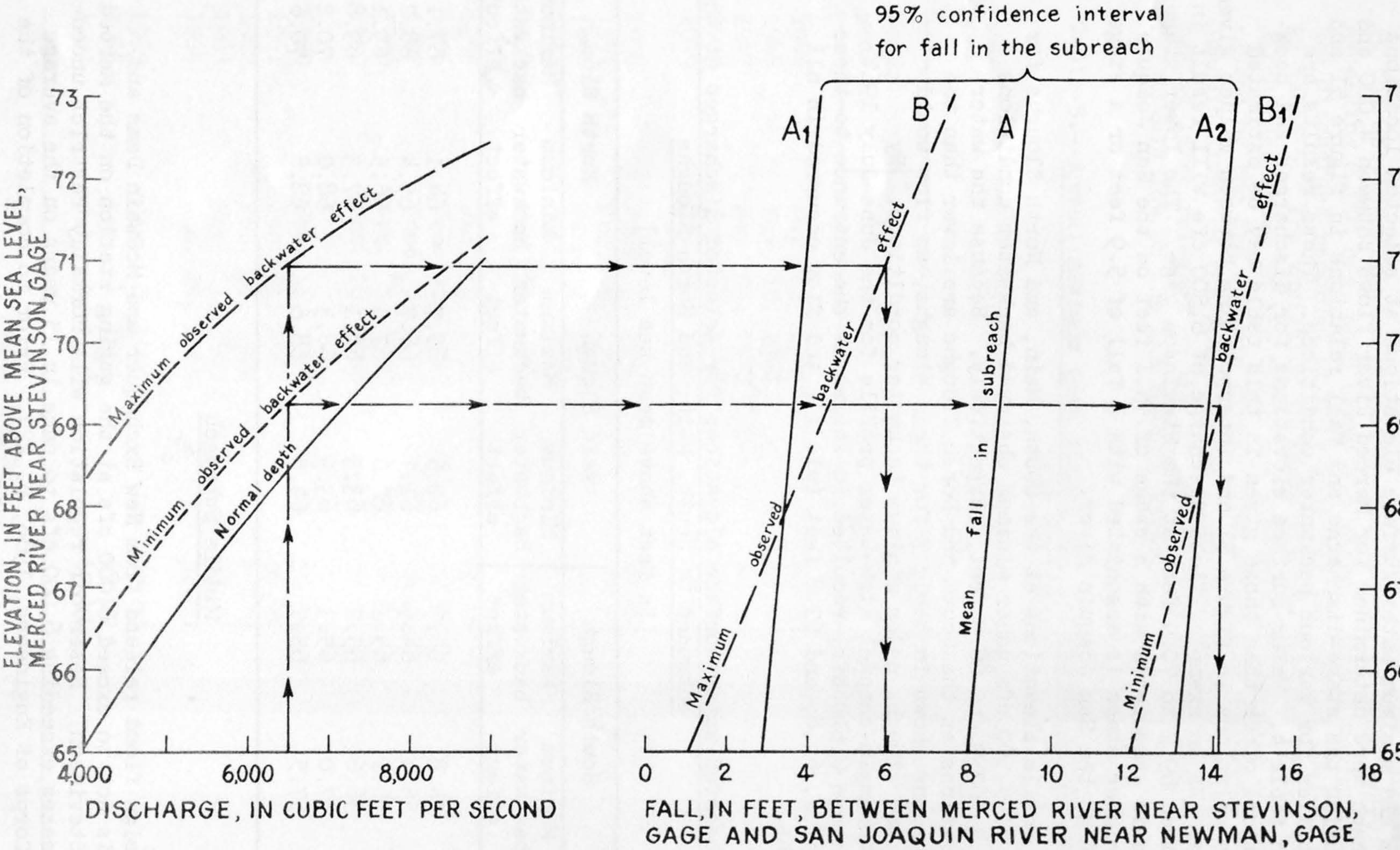


Figure 51._Discharge and fall relations at Merced River near Stevinson, gage.

Maximum and minimum water-surface elevations at selected locations in the subreach were determined for Merced River flows between 4,000 and 9,000 cfs, using the stage-discharge and fall relations in figure 51 and computed profiles for various backwater conditions. These results are tabulated in table 4. Water-surface elevations for discharges and backwater conditions other than those given in this table may be estimated by using the relations in figure 51 and interpolating between values given in the table. As an example, flood discharge of 6,500 cfs will result in a stage between 69.2 to 70.9 feet at the Stevinson gage. The lower stage at Stevinson is associated with a stage of 55.1 feet on the San Joaquin River; the higher stage is associated with a fall of 5.9 feet or a stage of 65.0 feet on the San Joaquin River.

Flood-profile elevations at the South, Main, and North Sloughs for a discharge of 6,500 cfs under minimum observed backwater conditions would be 56.3, 62.9, and 66.9 feet, respectively. Because the water-surface elevations at the South and North Sloughs are lower than the ground elevations shown in table 3 for these sloughs, no flow would enter these two sloughs under minimum backwater conditions. By comparison the main-channel high-water profile for the June-July 1967 flood, discharge 6,510 cfs, resulted in stages at the entrance to these sloughs of 61.2, 64.8, and 67.8 feet (pl. 1), and flow occurred in all three sloughs.

Table 4.--Observed water-surface elevations for selected discharges at the entrance to South, Main, and North Sloughs

[In feet above mean sea level]

Q (cfs)	South Slough		Main Slough		North Slough	
	Minimum backwater effect	Maximum backwater effect	Minimum backwater effect	Maximum backwater effect	Minimum backwater effect	Maximum backwater effect
4,000	54.8	64.7	60.6	65.7	64.1	67.1
5,000	55.4	64.9	61.7	66.7	65.4	68.2
6,000	56.1	65.1	62.6	67.5	66.5	69.1
7,000	56.6	65.2	63.3	68.1	67.3	69.8
8,000	57.0	65.3	63.9	68.3	68.0	70.2
9,000	57.5	65.5	64.5	68.6	68.8	70.6

Middle Subreach

The design flood release from New Exchequer and McSwain Dams and Reservoirs is not to exceed 6,000 cfs at the gaging station on the Merced River near Stevinson. Reservoir regulation will normally restrict uncontrolled releases exceeding 6,000 cfs to once in 15 years on the average (U.S. Army Corps of Engineers, 1968, p. 22). Prior to completion of the

New Exchequer Dam in 1967, annual maximum floodflows exceeding 4,000 cfs at the Stevinson gage occurred on the average of once every 2 years and exceeding 6,000 cfs, once every 4.5 years. Annual maximum peak stages and discharges recorded at this gage for the period 1945-67 are listed in table 5. The variation in stage for the various floods with similar discharges shown in the table are a result of variable backwater effect caused by the San Joaquin River. The effects of backwater from the San Joaquin River on the lower Merced River can be changed significantly by making large releases on the Merced River only when stages are relatively low on the San Joaquin River.

Table 5.--Annual maximum peak stages and discharges of the Merced River near Stevinson

[Modified after Young and Cruff, 1967, p. 122]

Water year	Flood date	Stage in feet, above mean sea level	Discharge (cfs)
1945	May 10, 1945	68.89	4,960
1946	May 10, 1946	67.67	4,050
1947	May 25, 1947	62.13	1,200
1948	June 11, 1948	67.90	4,210
1949	May 30, 1949	65.09	2,480
1950	June 3, 1950	67.00	3,450
1951	Dec. 5, 1950	73.79	13,600
1952	June 2, 1952	72.30	8,740
1953	Jan. 1, 1953	63.54	2,060
1954	May 21, 1954	66.19	3,480
1955	Jan. 19, 1955	60.17	912
1956	Dec. 28, 1955	72.84	11,200
1957	June 5, 1957	66.26	3,700
1958	Apr. 5, 1958	72.65	11,500
1959	Oct. 16, 1958	57.94	414
1960	Feb. 11, 1960	59.55	882
1961	Jan. 31, 1961	56.41	158
1962	Feb. 16, 1962	67.41	3,840
1963	May 11, 1963	67.37	4,610
1964	Jan. 24, 1964	57.60	260
1965	Jan. 8, 1965	72.09	11,000
1966	Dec. 4, 1965	67.60	4,780
1967	July 4, 1967	69.41	6,510

Water-surface elevations at the various cross sections (figs. 5-46) in the middle subreach, (between the gaging stations near Stevinson and at Shaffer Bridge), were determined by the step-backwater technique for discharges between 4,000 and 9,000 cfs and are tabulated in appendix A. These elevations are based on minimum observed backwater conditions at the Stevinson gage. Elevations of water surface for maximum observed backwater conditions are tabulated for cross sections 2 to 14 (stations 26,320 feet to 42,660 feet) in appendix B. For a given discharge, the profiles indicated in the two appendixes for cross sections 2-14 tend to converge in an upstream direction because the influence of backwater always decreases in the upstream direction. Upstream from cross section 14, backwater effect becomes negligible.

The computed water-surface profile for a 6,000 cfs discharge at the Stevinson gage, under minimum observed backwater conditions and channel conditions as determined at the time of the survey, indicates that most of this subreach will carry the design release within the natural and leveed banks. However, some lands adjacent to the river were flooded during the flood of June-July 1967 (discharge 6,510 cfs at the Stevinson gage) because backwater conditions, while relatively favorable, were not minimal at the time (pl. 1). Additional areas would have been inundated along the lower reaches of the Merced River downstream from cross section 14 if the San Joaquin River had reached a higher stage.

The computed water-surface profile for maximum observed backwater conditions indicates that overbank flow will first occur at cross section 12 (39,650 feet) whenever the discharge reaches about 6,500 cfs. For the reach farther upstream, computed and historical flood water-surface profiles indicate that a flow of 4,500 cfs at the Stevinson gage will cause overbank flooding at cross sections 22 (52,850 feet) and 69 (98,220 feet), (pls. 1, 2, figs. 11, 25). Few additional sites will be inundated when flows increase from 4,500 to 8,000 cfs as shown on the profiles (pls. 1-5) and cross-section plots (figs. 5-46).

Accuracy of the computed profiles in this subreach was evaluated by comparing the flood of June-July 1967 (discharge 6,510 cfs at the Stevinson gage) with the corresponding computed profile. Differences between the measured and computed profiles averaged -0.1 foot at the 141 cross sections used in the step-backwater computation. These differences were 0.4 foot or less at 91 percent of the cross sections and the maximum deviation at any cross section was 0.8 foot. In addition, the computed profiles were compared with stage-discharge relations that have been determined by discharge measurements for gaging stations at cross sections 121 (145,740 feet) and 141 (171,200 feet). A comparison of the measured and computed stage-discharge relations at the cross sections is given in table 6.

Table 6.--Comparison of measured and computed stage-discharge relations

Gaging station	Discharge (cfs)	Measured stage (feet)	Computed stage (feet)
Merced River at	4,000	104.5	104.2
Cressey (DWR)	6,000	106.9	106.7
(cross section 121)	6,850 (June-July 1967 flood)	107.7	107.5
	9,000	109.6	109.6
Merced River	4,000	124.7	124.6
at Shaffer	6,000	126.1	126.4
Bridge near	6,900 (June-July	126.7	127.3
Cressey (cross	1967 flood)		
section 141)	9,000	127.8	128.3

The close agreement between measured and computed stages at these gages indicates that water-surface elevations determined at the cross sections for discharges other than those used in the comparison, will probably be of similar accuracy.

Most of the levees in this subreach are constructed of earth (sandy loam) fill, and damage or scour will occur, especially during periods of sustained releases. In the lower part of the subreach prolonged inundation caused by backwater from the San Joaquin River also may cause levee and bank scour. Locations which showed significant damage due to scouring at the time of the survey are shown on plates 1-4. Most of the scour damage areas are located between cross sections 2 (26,320 feet) and 75 (103,890 feet).

Gravel plants are operated in this subreach near cross sections 119 (142,840 feet) and 131 (159,420 feet). At present, these operations have not altered the carrying capacity of the channel.

Dry Creek enters the Merced River from the north near cross section 138 (167,400 feet, pl. 3) and is the only tributary of significant size in this reach. Runoff from this watershed results primarily from rainstorms that occur during the winter months between November and April. At present, streamflow from this watershed is unregulated.

Between September 1965 (when a gaging station was installed on Dry Creek 18.7 miles upstream from its confluence with the Merced River) and April 1968, 11 peak flows exceeding 1,000 cfs were recorded. The maximum recorded discharge during this period was 4,890 cfs on April 27, 1967. Unfortunately, flood frequency relations (Young and Cruff, 1967, p. 12) for this basin are unavailable because of a lack of records for unregulated streams in the Central Valley region. However, historical information from local residents indicates that larger floods on Dry Creek have occurred in the past.

Floods from Dry Creek cause increased flow in the Merced River, but significantly large peaks on Dry Creek seldom occur during high flows on the Merced River. Furthermore, as shown in figure 52, the peak discharges of Dry Creek are reduced by about 50 percent as the flood traverses the reach between the gaging stations on Dry Creek near Snelling and on the Merced River near Cressey. Scatter of the points in figure 52 results from differences in flood duration and variable channel storage from earlier flows in Dry Creek and the Merced River. The time of flood travel between the Dry Creek gage and the mouth is about 12 hours, and to the Cressey gage, about 15 hours. The influence of Dry Creek runoff on Merced River flows is barely noticeable at the Stevinson gage.

Upper Subreach

Water-surface profiles shown on plates 4 and 5 for the upper subreach between the gaging stations at Shaffer Bridge and below Merced Falls were determined by using floodmarks left by the June-July 1967 flood. The upper subreach is characterized by a meandering main channel and a wide flood plain dissected by four sloughs. Dana and Hopeton Sloughs (pls. 4 and 5) are tributary to Ingalsbe Slough which enters the main channel downstream from cross section 142 (191,000 feet).

Ingalsbe Slough used to leave the main channel near Snelling, and the entrance to Hopeton Slough probably is just downstream from cross section 144 (244,820 feet). The entrances to these sloughs and the main-channel discharge at which flows begin are difficult to determine because the slough inlets have been obstructed by dredge tailings. Most of the flow in these sloughs is caused by irrigation drainage water, and seepage through the dredge tailings during flood stages in the main channel.

Ehe entrance to Dana Slough is located about 1 mile downstream from cross section 144 (244,820 feet) and now serves as the inlet for the Cowell Ditch (California Dept. of Water Resources, 1966). Flows in this ditch may be controlled at the inlet structure so that discharges in Dana Slough are normally only a minor part of the main-channel discharge.

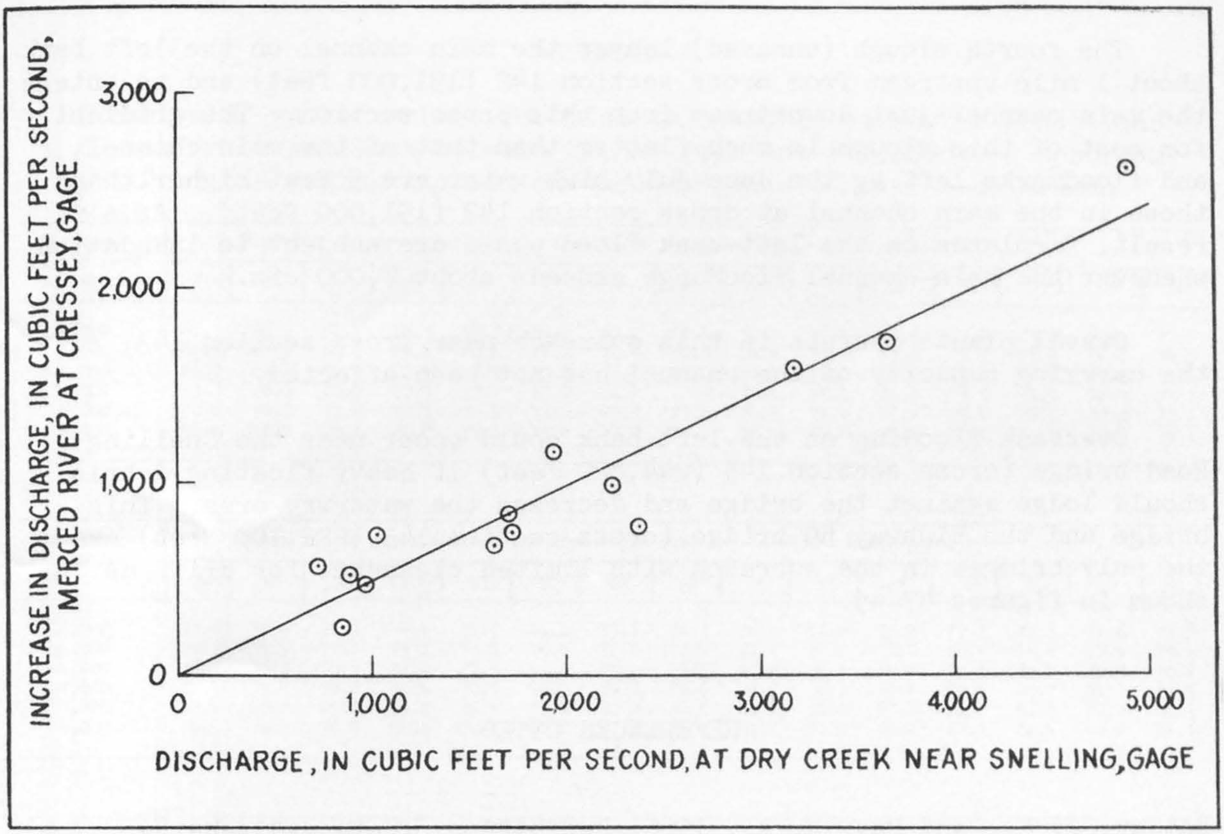


Figure 52.—Effect of Dry Creek flow on discharge of the Merced River at Cressey.

The fourth slough (unnamed) leaves the main channel on the left bank about 1 mile upstream from cross section 142 (191,000 feet) and re-enters the main channel just downstream from this cross section. The gradient for most of this slough is much flatter than that of the main channel, and floodmarks left by the June-July high water are 5 feet higher than those in the main channel at cross section 142 (191,000 feet). As a result, farmlands on the left-bank flood plain are subject to inundation whenever the main-channel discharge exceeds about 7,000 cfs.

Gravel plants operate in this subreach near cross section 143, but the carrying capacity of the channel has not been affected.

Overbank flooding on the left bank could occur near the Snelling Road bridge (cross section 145 (244,820 feet) if heavy floating debris should lodge against the bridge and decrease the waterway area. This bridge and the Highway 59 bridge (cross section 143 (222,100 feet) are the only bridges in the subreach with limited clearance for drift as shown in figures 47-49.

REFERENCES CITED

Bailey, J. F., and Ray, H. A., 1966, Definition of stage-discharge relation in natural channels by step-backwater analysis: U.S. Geol. Survey Water-Supply Paper 1869-A, 24 p.

California Department of Water Resources, 1967, Hydrologic data: 1966 Volume IV: San Joaquin Valley: California Dept. Water Resources Bull. 130-66, 455 p.

Chow, V. T., 1959, Open-channel hydraulics: New York, McGraw-Hill Book Co., 680 p.

Davis, G. H., Green, J. H., Olmsted, F. H., and Brown, D. W., 1959, Ground-water conditions and storage capacity in the San Joaquin Valley, California: U.S. Geol. Survey Water-Supply Paper 1469, 287 p.

U.S. Army Corps of Engineers, 1968, New Exchequer Dam & Reservoir, Merced River, California, Report on reservoir regulation for flood control: U.S. Army Corps of Engineers, open-file report, 31 p. (February).

Young, L. E., and Cruff, R. W., 1967, Magnitude and frequency of floods in the United States: U.S. Geol. Survey Water-Supply Paper 1686, pt. 2, v. 2, 303 p.

APPENDIX A

Elevation of water surface for minimum measured backwater conditions at cross sections between 26,317 feet and 171,200 feet upstream from the gaging station on the San Joaquin River near Newman

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
4,000	66.3	66.5	67.3	68.3	68.7	69.0	69.2	69.6	70.1	70.5	70.8	71.2	71.9	73.3	73.5
5,000	67.7	67.9	68.6	69.4	69.8	70.1	70.4	70.9	71.4	71.8	72.1	72.5	73.3	74.7	74.8
6,000	68.9	69.1	69.7	70.4	70.8	71.2	71.4	72.0	72.4	72.9	73.2	73.6	74.4	75.9	76.1
7,000	69.8	70.0	70.6	71.3	71.7	72.1	72.4	72.9	73.4	73.9	74.2	74.6	75.5	77.0	77.1
8,000	70.6	70.9	71.4	72.1	72.6	73.0	73.3	73.8	74.4	74.9	75.2	75.7	76.5	78.0	78.1
9,000	71.4	71.6	72.1	72.8	73.3	73.8	74.1	74.7	75.2	75.8	76.1	76.6	77.4	78.9	79.1

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
4,000	73.8	74.1	74.3	74.4	74.7	75.1	75.6	76.1	76.4	76.6	76.9	77.1	77.3	77.6	77.8
5,000	75.2	75.5	75.7	75.8	76.1	76.5	77.0	77.4	77.7	78.0	78.2	78.4	78.6	78.9	79.1
6,000	76.4	76.7	76.9	77.0	77.3	77.7	78.1	78.5	78.8	79.1	79.4	79.4	79.6	80.0	80.2
7,000	77.4	77.7	78.0	78.2	78.5	78.9	79.2	79.6	79.9	80.1	80.4	80.5	80.7	81.1	81.3
8,000	78.4	78.7	79.0	79.2	79.5	79.9	80.2	80.6	80.8	81.1	81.4	81.4	81.6	82.0	82.3
9,000	79.4	79.7	79.9	80.1	80.4	80.8	81.1	81.4	81.7	81.9	82.2	82.2	82.4	82.8	83.1

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
4,000	78.1	78.6	78.8	79.0	79.4	79.5	79.7	79.9	80.1	80.4	80.6	80.9	81.1	81.4	81.9
5,000	79.4	80.0	80.2	80.5	80.8	81.0	81.1	81.4	81.6	81.8	82.1	82.3	82.6	82.8	83.3
6,000	80.5	81.1	81.3	81.6	81.9	82.1	82.2	82.5	82.7	83.0	83.2	83.5	83.8	84.0	84.4
7,000	81.6	82.2	82.4	82.7	83.0	83.1	83.3	83.6	83.8	84.1	84.4	84.6	84.9	85.1	85.6
8,000	82.5	83.1	83.4	83.6	84.0	84.1	84.3	84.6	84.8	85.1	85.3	85.7	85.9	86.2	86.7
9,000	83.3	84.0	84.2	84.5	84.8	85.0	85.2	85.5	85.7	86.0	86.3	86.6	86.9	87.2	87.6

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61
4,000	82.1	82.5	82.6	83.0	83.4	83.8	84.4	84.7	85.1	85.4	85.6	86.0	86.6	86.9	87.2
5,000	83.5	83.8	84.0	84.4	84.8	85.3	85.8	86.1	86.5	86.8	87.0	87.5	88.1	88.5	88.7
6,000	84.7	85.0	85.2	85.6	86.0	86.5	86.9	87.2	87.6	88.0	88.2	88.6	89.2	89.6	89.9
7,000	85.8	86.1	86.3	86.6	87.1	87.5	87.9	88.2	88.6	89.0	89.2	89.6	90.2	90.6	90.9
8,000	86.9	87.2	87.4	87.6	88.1	88.5	88.9	89.2	89.6	89.9	90.2	90.6	91.1	91.5	91.8
9,000	87.9	88.2	88.4	88.6	89.0	89.4	89.8	90.1	90.4	90.8	91.1	91.5	92.0	92.4	92.7

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76
4,000	87.3	87.4	87.6	87.9	88.3	88.5	88.6	88.9	89.3	89.5	89.7	90.0	90.2	90.4	90.5
5,000	88.9	88.9	89.1	89.4	89.9	90.1	90.2	90.5	90.9	91.1	91.3	91.6	91.8	92.0	92.1
6,000	90.0	90.1	90.3	90.6	91.1	91.3	91.4	91.7	92.0	92.2	92.5	92.8	93.1	93.3	93.4
7,000	91.0	91.1	91.3	91.6	92.1	92.3	92.4	92.6	92.9	93.2	93.5	93.8	94.1	94.3	94.5
8,000	91.9	92.0	92.2	92.6	93.0	93.2	93.3	93.6	93.8	94.1	94.4	94.8	95.1	95.2	95.4
9,000	92.8	92.9	93.1	93.4	93.9	94.1	94.2	94.4	94.7	95.0	95.2	95.7	96.0	96.1	96.3

Appendix A.--Water surface for minimum measured backwater--Continued

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91
4,000	90.8	91.0	91.2	91.6	92.0	92.1	92.4	92.5	92.7	93.0	93.2	93.6	94.0	94.5	95.1
5,000	92.4	92.7	92.8	93.3	93.6	93.8	94.0	94.1	94.3	94.6	94.9	95.3	95.7	96.2	96.7
6,000	93.7	94.0	94.2	94.6	95.0	95.1	95.4	95.5	95.7	96.0	96.3	96.6	97.0	97.4	98.0
7,000	94.8	95.1	95.3	95.8	96.1	96.3	96.6	96.7	96.9	97.2	97.5	97.9	98.2	98.6	99.1
8,000	95.8	96.1	96.3	96.8	97.1	97.3	97.6	97.8	98.0	98.3	98.6	98.9	99.3	99.7	100.2
9,000	96.7	97.0	97.2	97.8	98.1	98.3	98.6	98.8	99.0	99.4	99.6	100.0	100.3	100.7	101.2

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	92	93	94	95	96	97	97A	98	99	100	101	102	103	104	105
4,000	95.4	95.6	96.0	96.5	96.8	97.1	97.2	97.4	97.4	97.6	97.7	97.8	98.3	98.8	99.5
5,000	96.9	97.2	97.6	98.0	98.4	98.7	98.8	99.0	99.0	99.2	99.3	99.4	99.7	100.2	100.8
6,000	98.2	98.5	98.9	99.4	99.7	100.1	100.2	100.3	100.4	100.5	100.6	100.7	101.0	101.4	102.0
7,000	99.4	99.6	100.0	100.5	100.9	101.2	101.3	101.5	101.6	101.8	101.8	101.9	102.2	102.6	103.2
8,000	100.4	100.7	101.1	101.6	102.0	102.2	102.3	102.5	102.6	102.8	102.8	102.9	103.2	103.6	104.1
9,000	101.5	101.8	102.2	102.6	103.0	103.3	103.3	103.5	103.6	103.8	103.8	103.9	104.2	104.6	105.1

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
4,000	99.8	100.0	100.3	100.6	100.9	101.2	101.7	101.8	102.2	102.6	102.8	103.0	103.3	103.3	103.9
5,000	101.1	101.3	101.5	101.8	102.2	102.4	102.8	103.0	103.4	103.8	104.0	104.2	104.5	104.5	105.2
6,000	102.3	102.5	102.7	103.1	103.4	103.6	104.0	104.1	104.5	104.9	105.1	105.3	105.7	105.8	106.4
7,000	103.4	103.6	103.8	104.1	104.4	104.6	105.0	105.1	105.6	105.9	106.2	106.3	106.8	106.8	107.4
8,000	104.3	104.5	104.8	105.1	105.4	105.6	105.9	106.0	106.5	106.9	107.1	107.3	107.8	107.8	108.3
9,000	105.3	105.4	105.7	106.0	106.3	106.5	106.8	106.9	107.4	107.8	108.0	108.1	108.7	108.7	109.2

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section														
	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135
4,000	104.2	105.1	106.1	107.4	108.9	110.0	112.0	113.1	114.1	115.2	115.5	115.5	115.6	117.7	118.7
5,000	105.5	106.4	107.3	108.3	109.8	111.0	113.0	114.2	115.0	116.0	116.3	116.3	116.5	118.4	119.3
6,000	106.7	107.5	108.3	109.3	110.6	111.8	113.9	115.1	115.9	116.8	117.1	117.1	117.3	119.0	119.8
7,000	107.7	108.5	109.3	110.2	111.5	112.6	114.8	116.0	116.7	117.5	117.8	117.9	118.0	119.6	120.3
8,000	108.7	109.5	110.2	111.0	112.3	113.5	115.6	116.9	117.6	118.2	118.6	118.6	118.8	120.3	120.9
9,000	109.6	110.4	111.0	111.7	113.0	114.2	116.4	117.6	118.3	118.9	119.3	119.3	119.4	120.9	121.4

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section					
	136	137	138	139	140	141
4,000	120.3	122.1	122.6	122.9	123.2	124.6
5,000	120.8	122.6	123.2	123.5	123.9	125.6
6,000	121.3	123.0	123.6	124.0	124.6	126.4
7,000	121.7	123.5	124.0	124.4	125.3	127.3
8,000	122.2	123.9	124.5	124.9	125.9	127.8
9,000	122.6	124.2	124.8	125.3	126.4	128.3

APPENDIX B

Elevation of water surface for maximum measured backwater conditions at cross sections between 26,317 feet and 42,660 feet upstream from the gaging station on the San Joaquin River near Newman

Discharge (cubic feet per second)	Elevation of water surface (in feet above mean sea level) at cross section												
	2	3	4	5	6	7	8	9	10	11	12	13	14
4,000	68.4	68.5	68.9	69.4	69.6	69.8	70.0	70.4	70.7	71.1	71.3	71.7	72.1
5,000	69.6	69.8	70.1	70.5	70.8	71.0	71.2	71.6	72.0	72.4	72.6	73.0	73.4
6,000	70.5	70.6	71.0	71.4	71.8	72.0	72.2	72.7	73.0	73.4	73.7	74.1	74.5
7,000	71.3	71.5	71.8	72.3	72.6	72.9	73.2	73.6	74.0	74.5	74.8	75.2	75.7
8,000	71.9	72.1	72.4	72.9	73.3	73.7	74.0	74.4	74.9	75.4	75.7	76.1	76.6
9,000	72.4	72.6	73.0	73.5	74.0	74.3	74.6	75.1	75.6	76.1	76.4	76.9	77.4

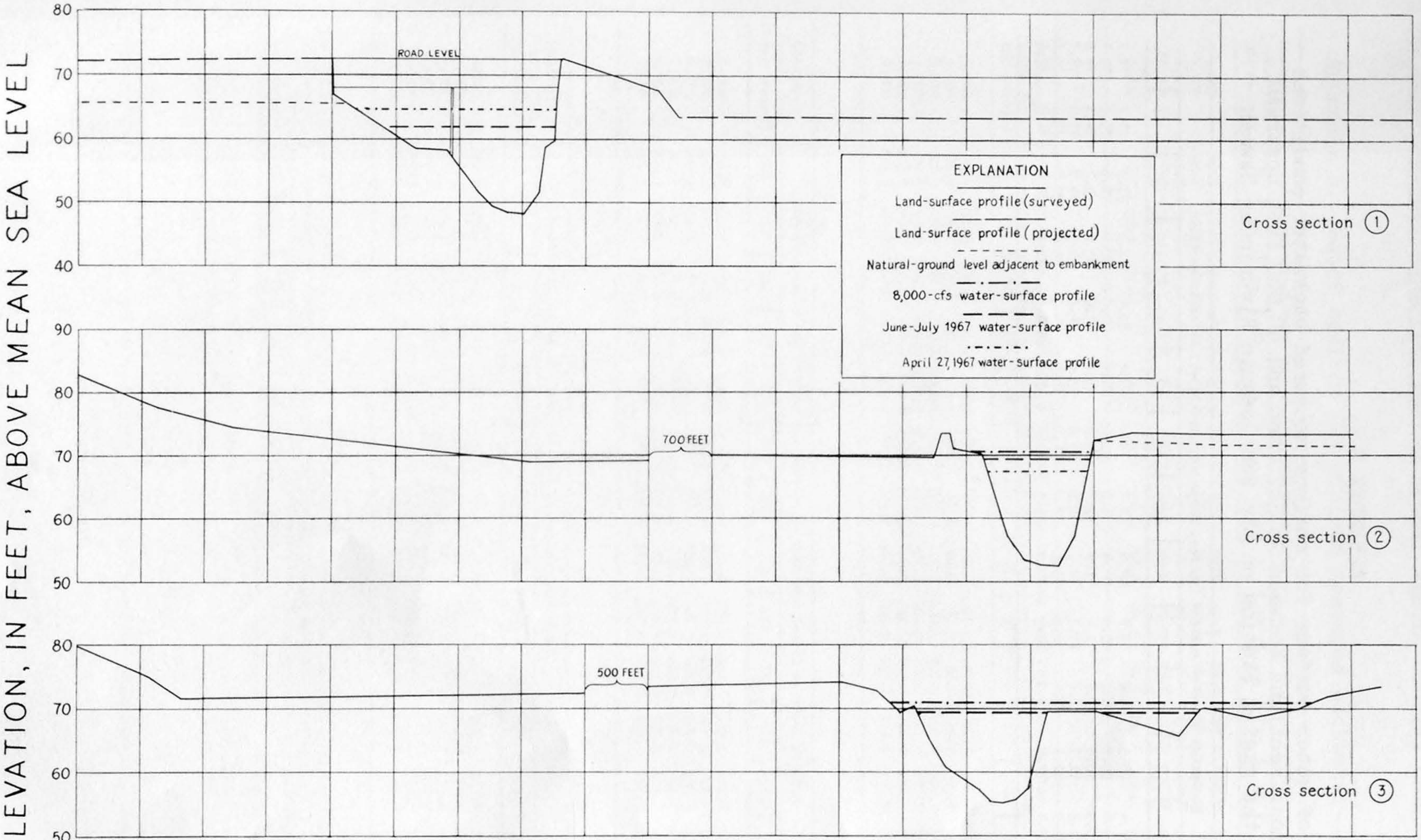


Figure 5.—Cross sections 1-3

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

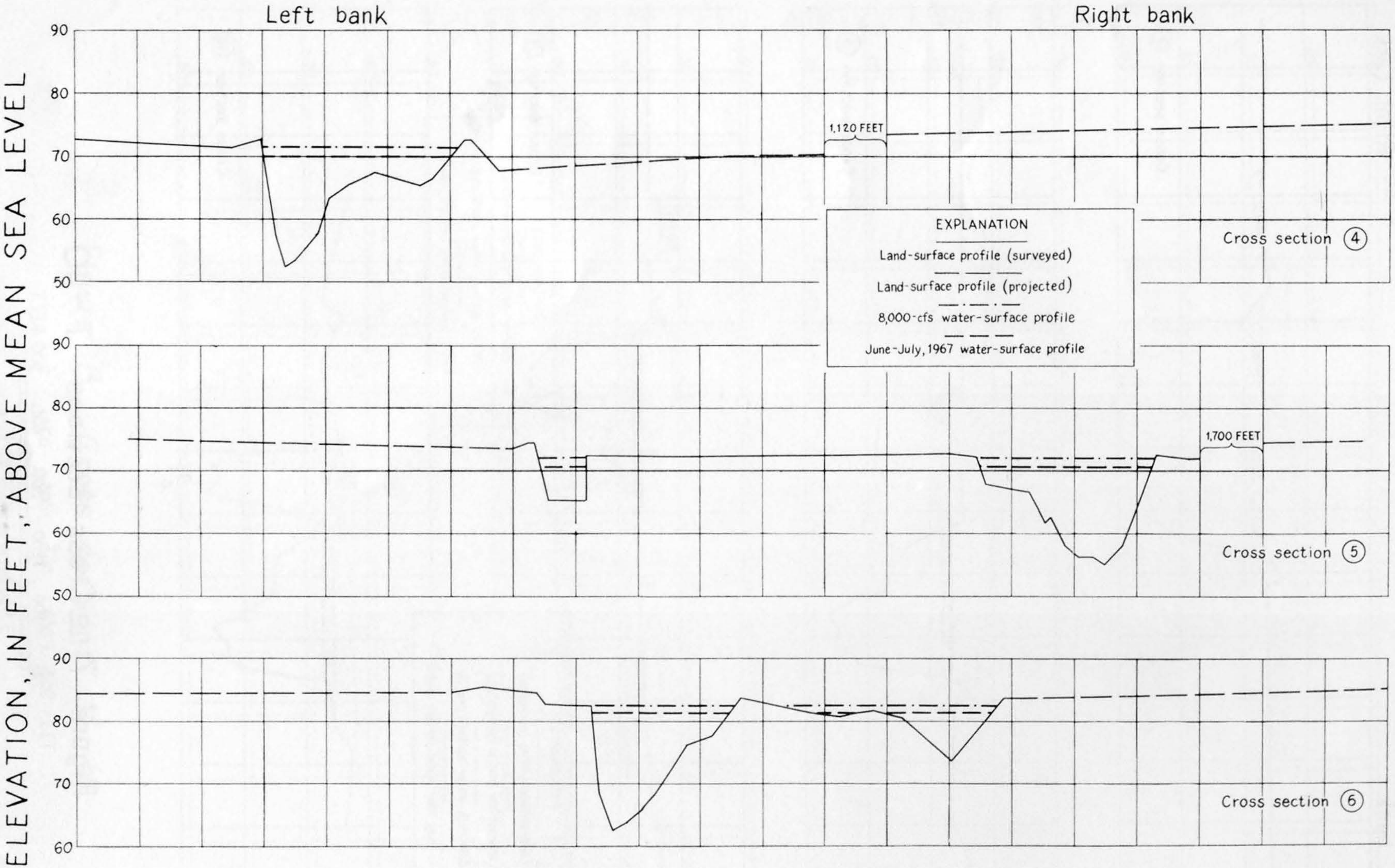


Figure 6 .—Cross sections 4 - 6

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

U.S. GEOLOGICAL SURVEY

FIGURE 6

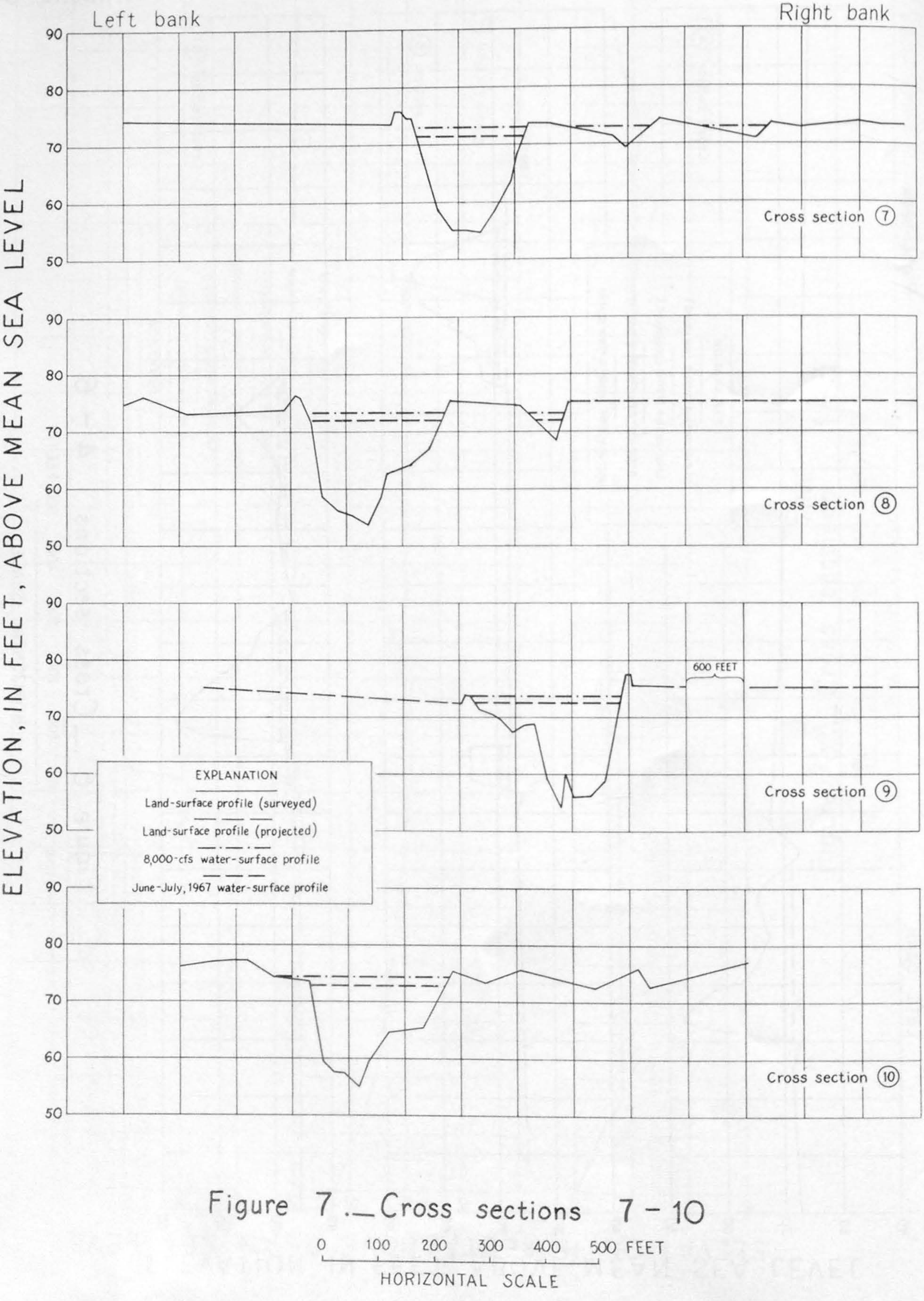


Figure 7.—Cross sections 7 - 10

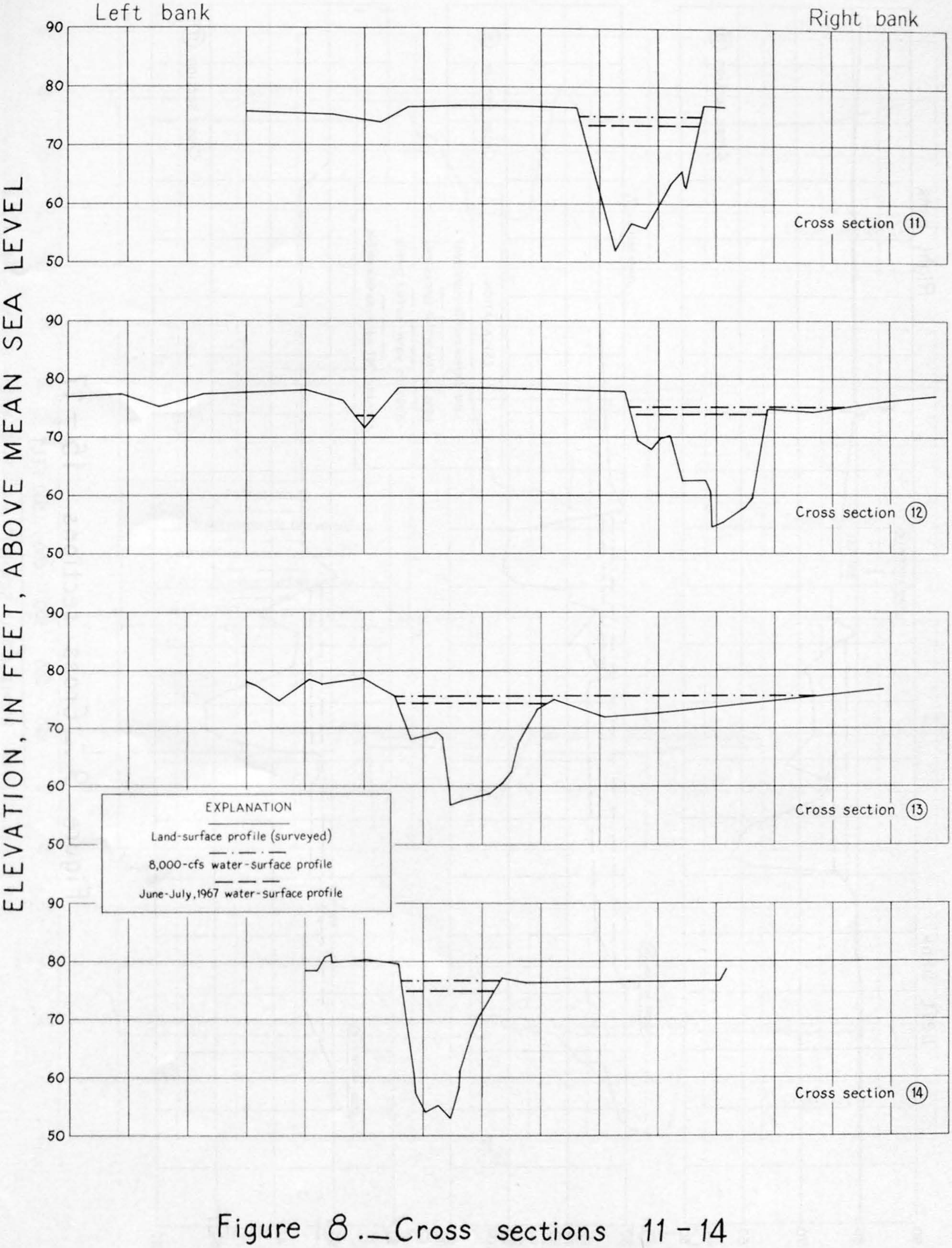


Figure 8.—Cross sections 11 - 14

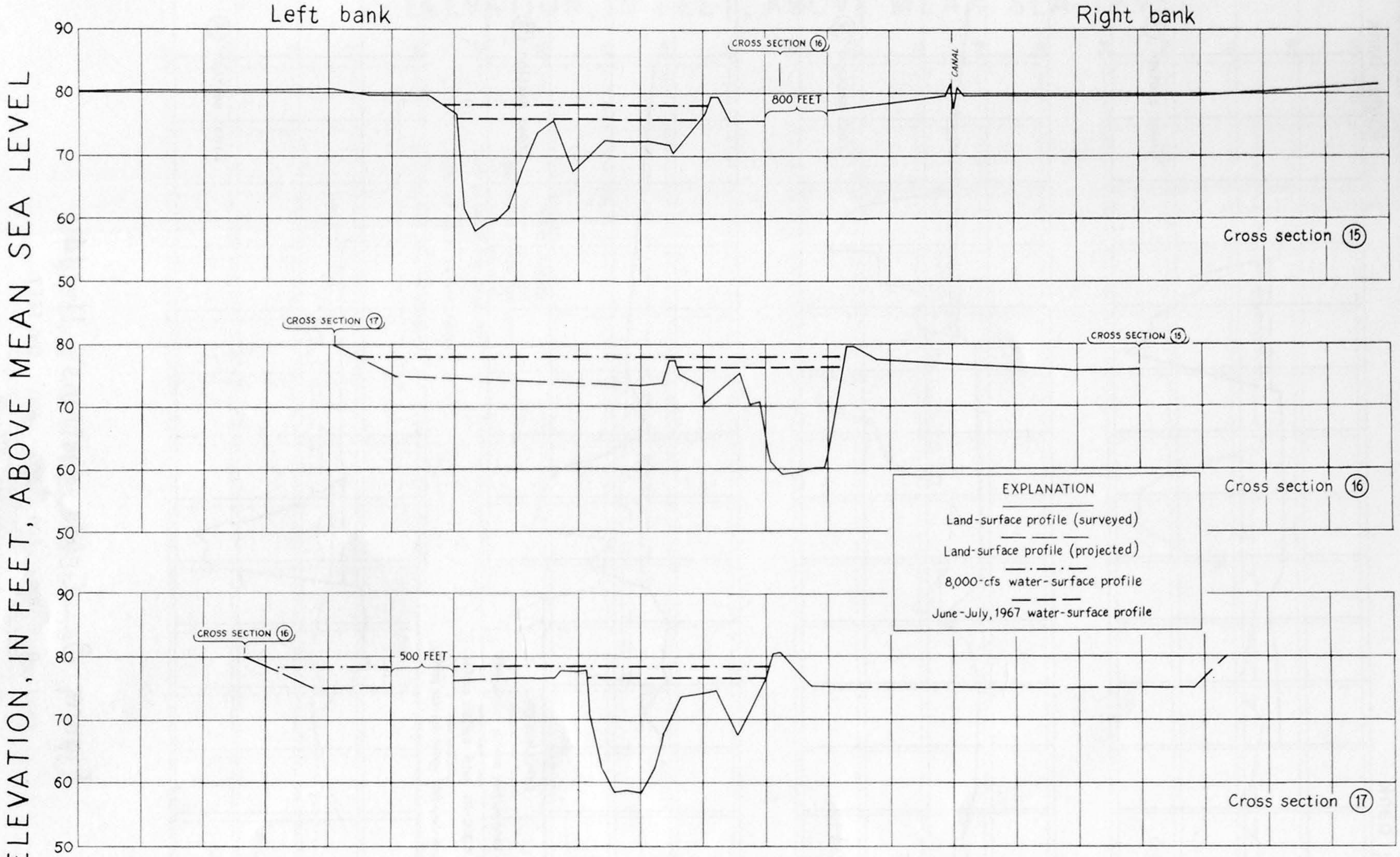


Figure 9.—Cross sections 15-17

0 100 200 300 400 500 FEET
HORIZONTAL SCALE

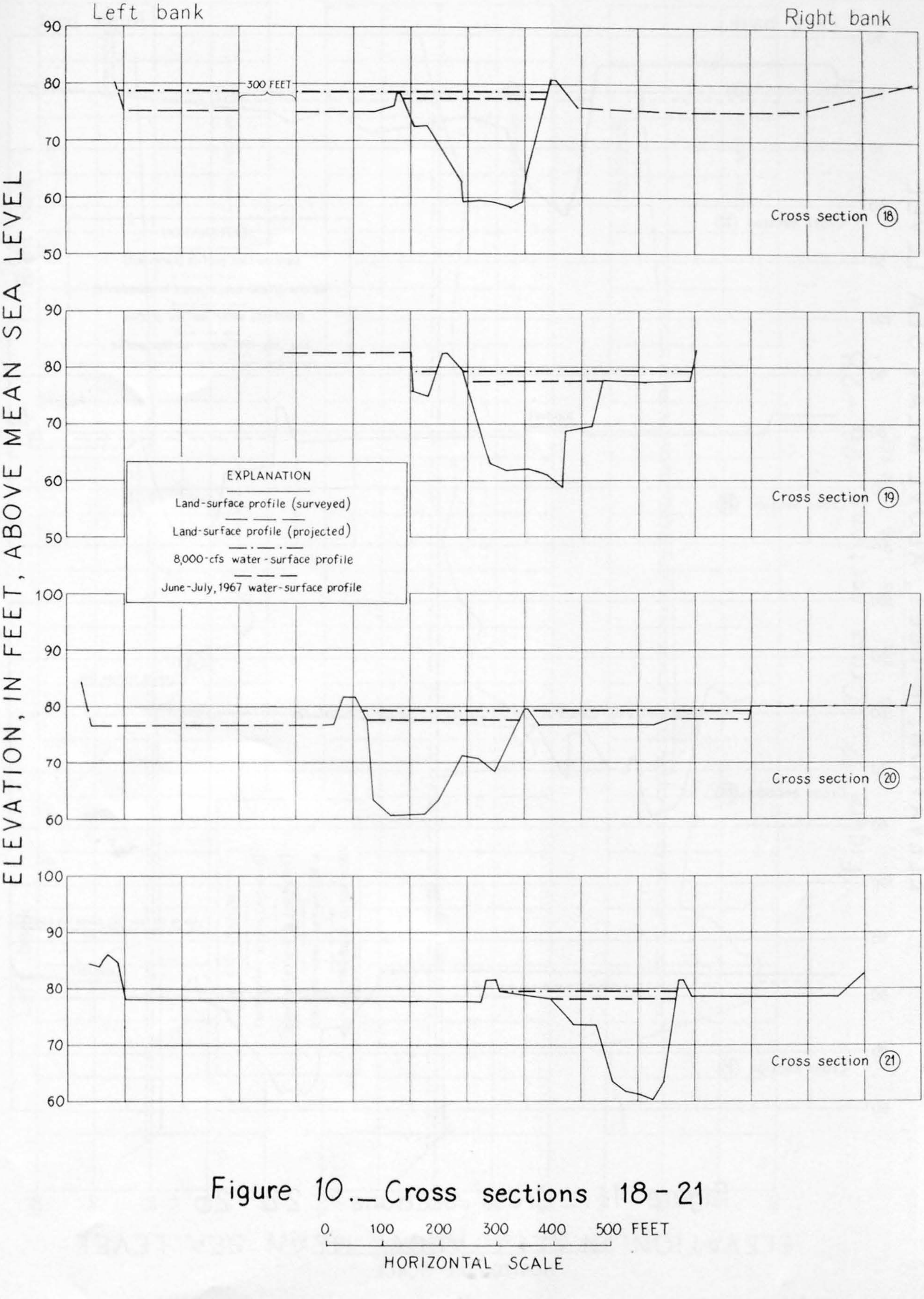


Figure 10.—Cross sections 18-21

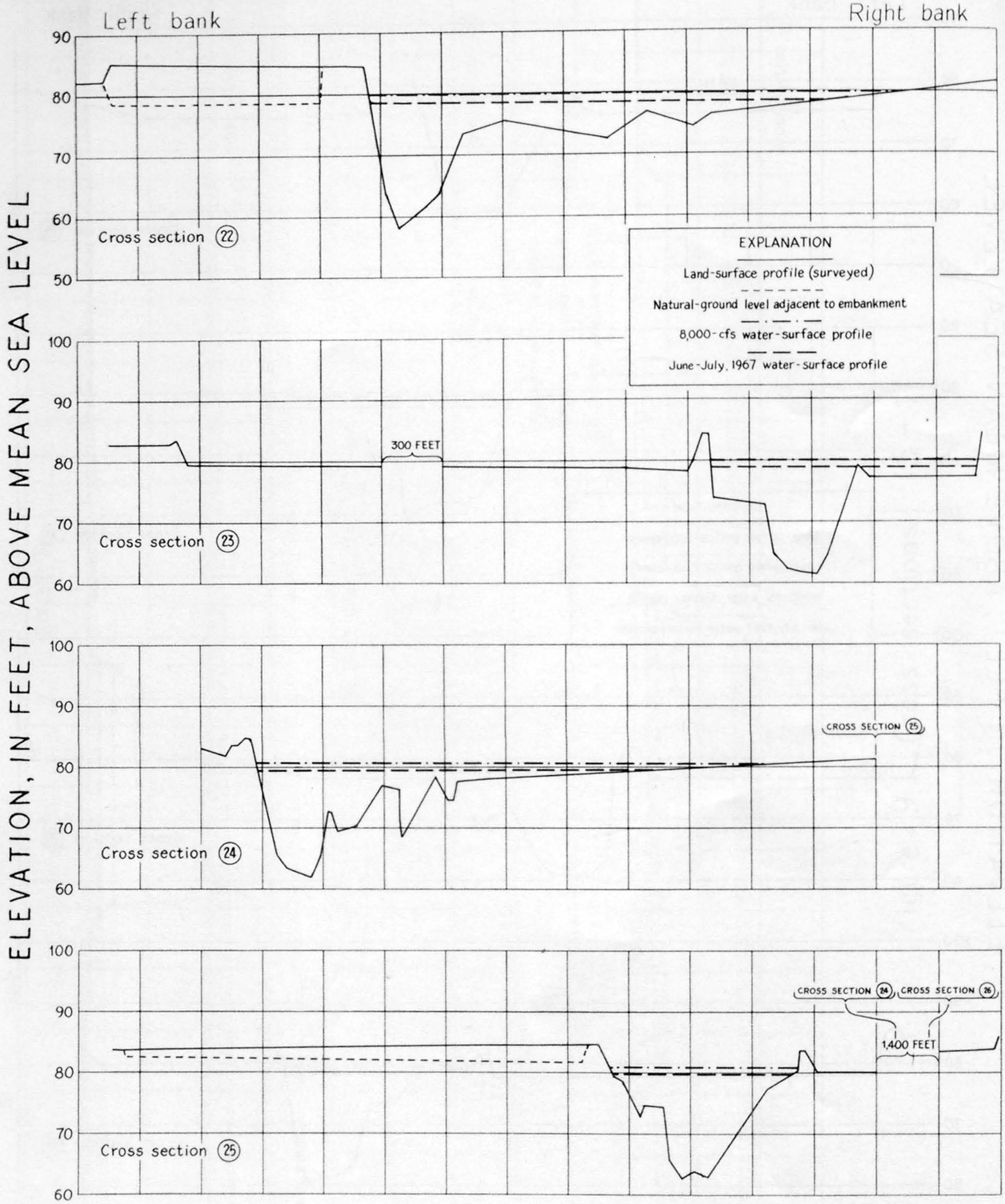


Figure 11.—Cross sections 22-25

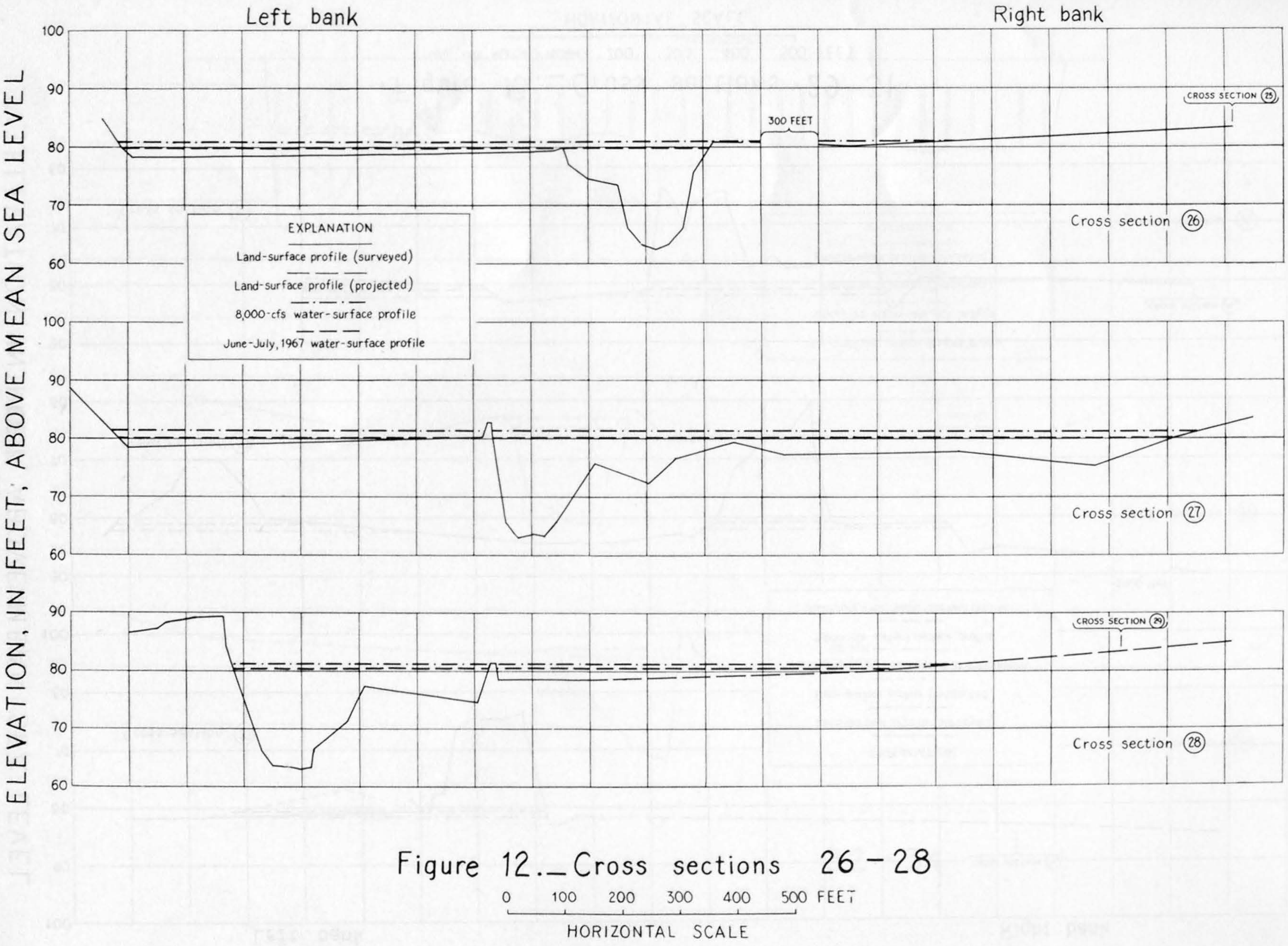


Figure 12.—Cross sections 26–28

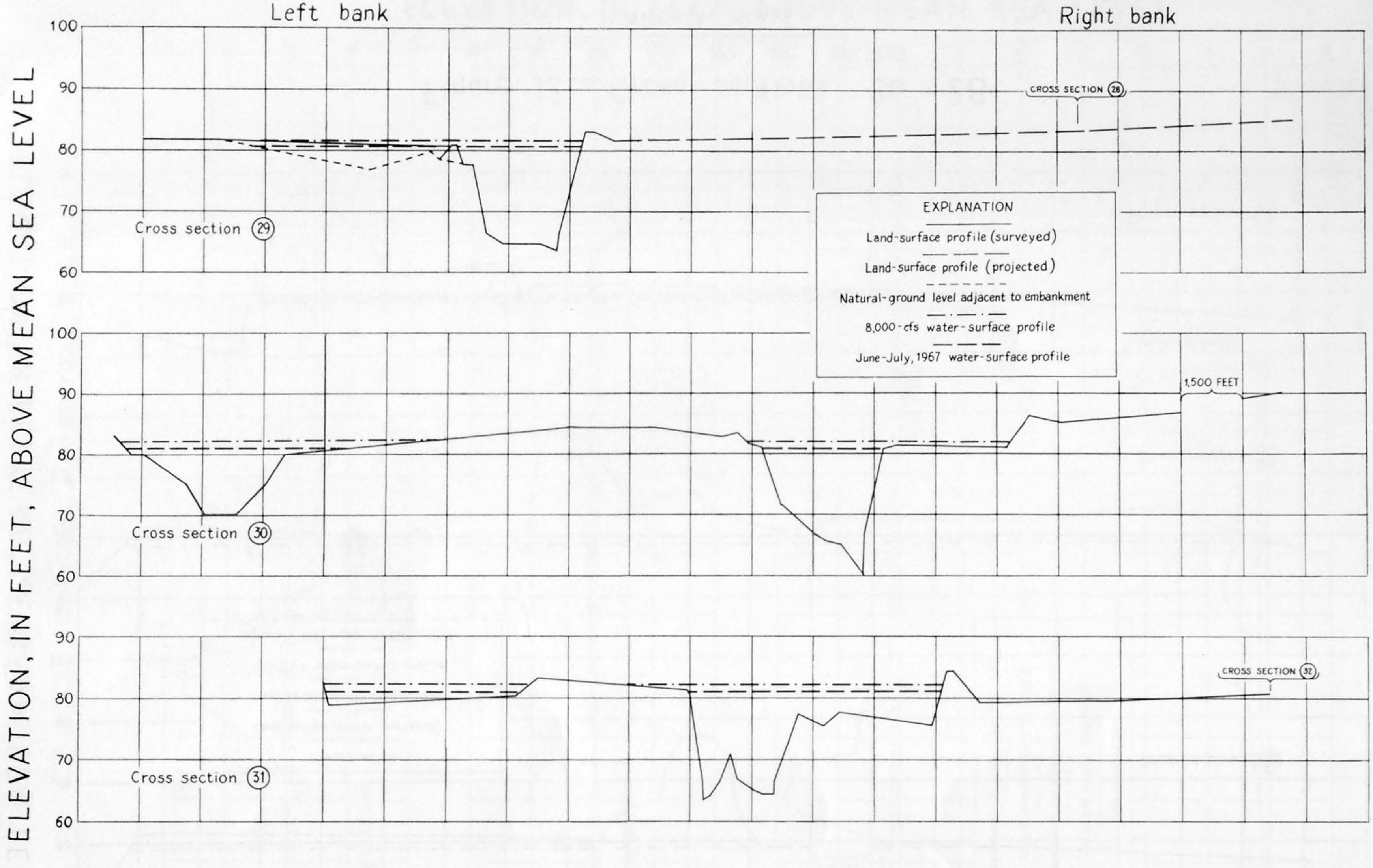


Figure 13.—Cross sections 29-31

0 100 200 300 400 500 FEET
HORIZONTAL SCALE

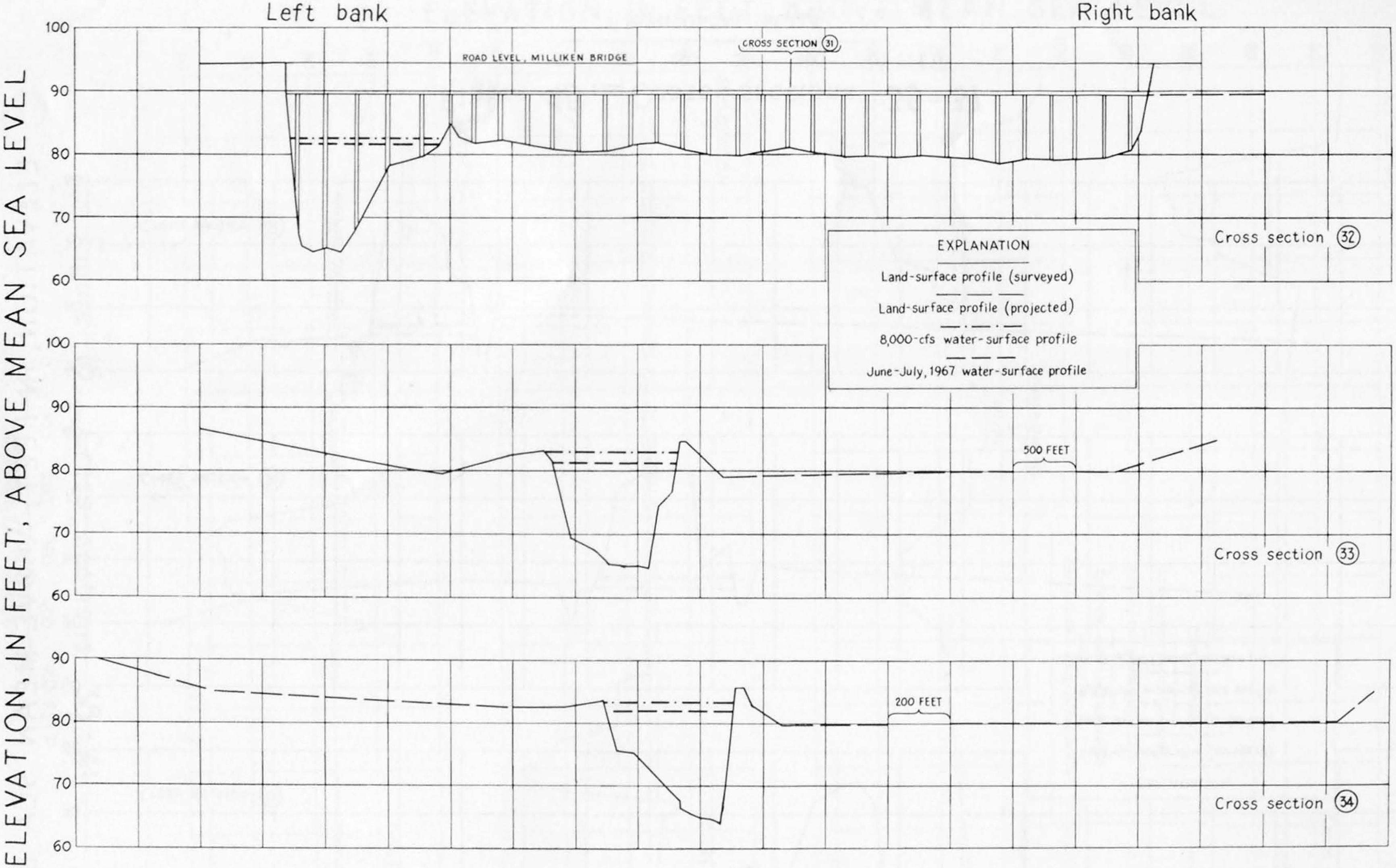


Figure 14.—Cross sections 32–34

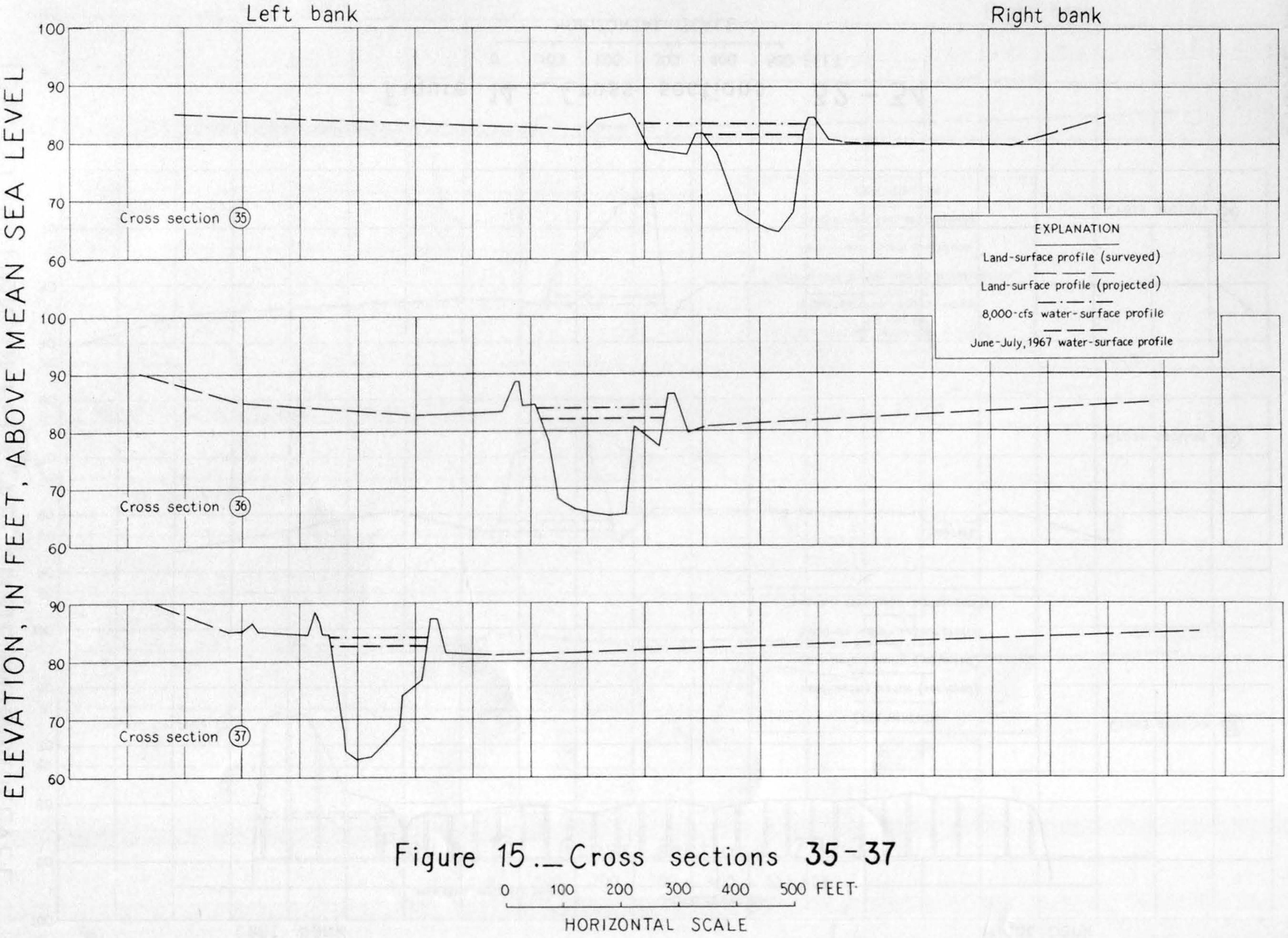


Figure 15.—Cross sections 35-37

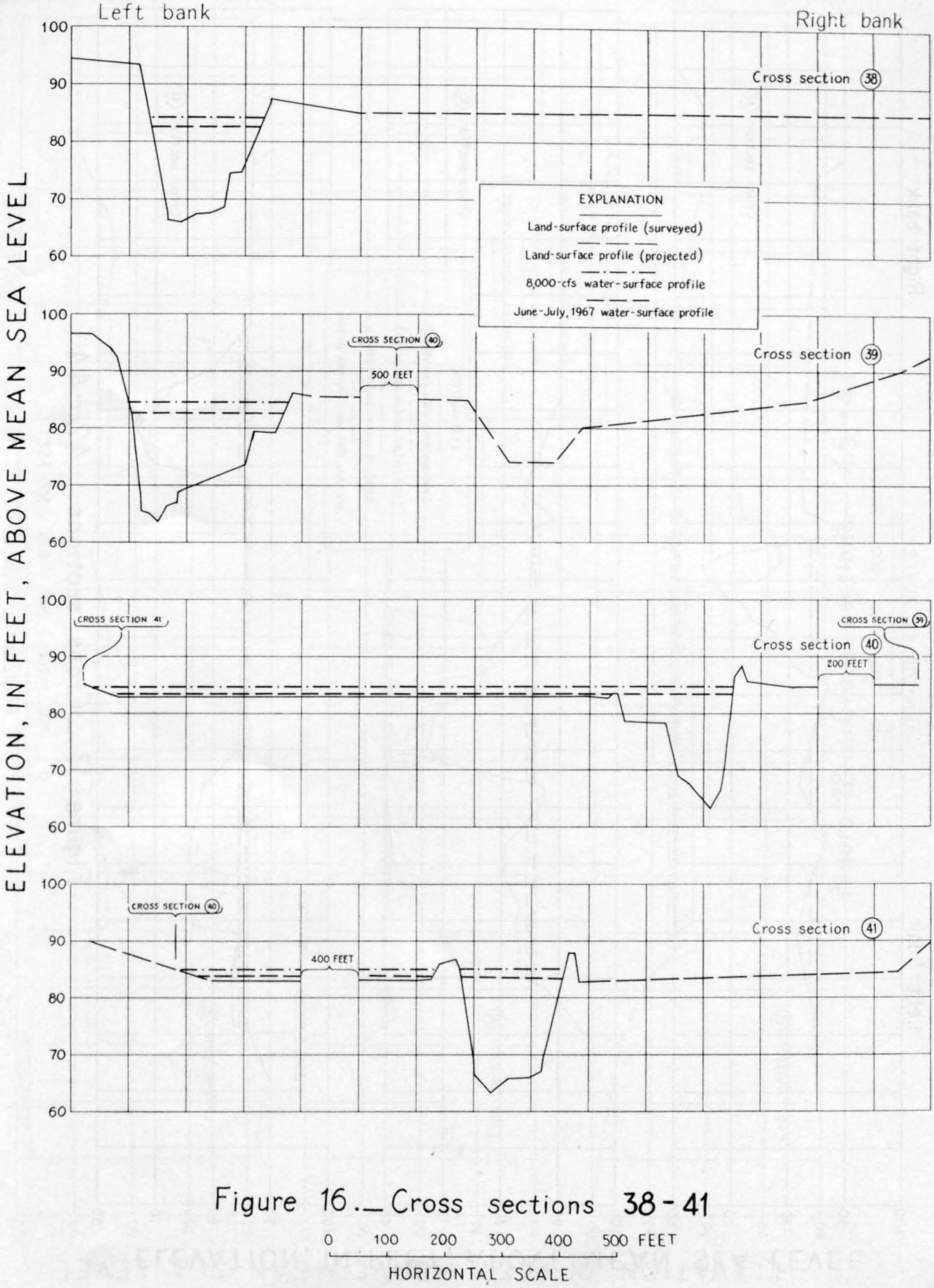


Figure 16.—Cross sections 38-41

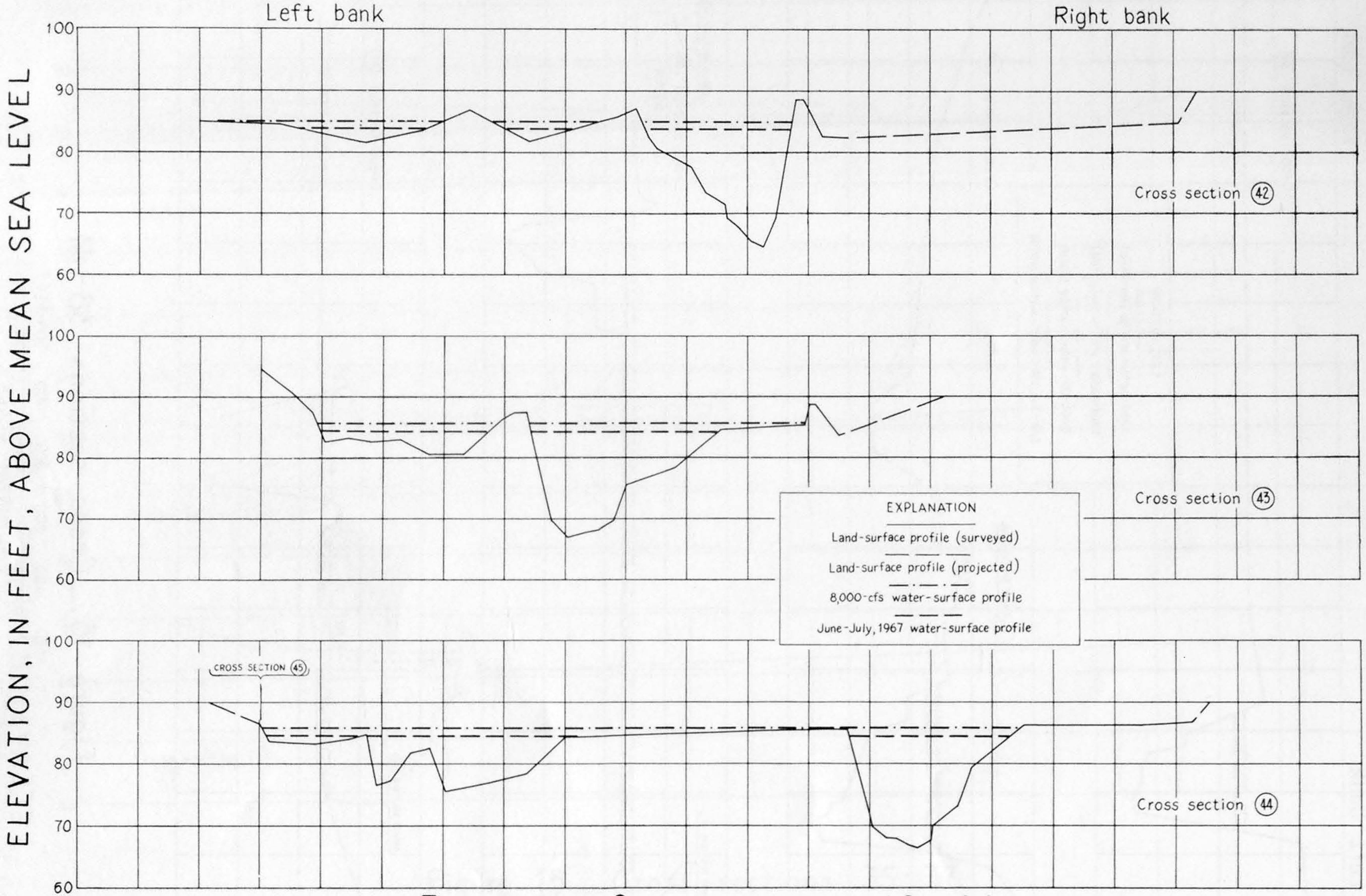


Figure 17.—Cross sections 42-44

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

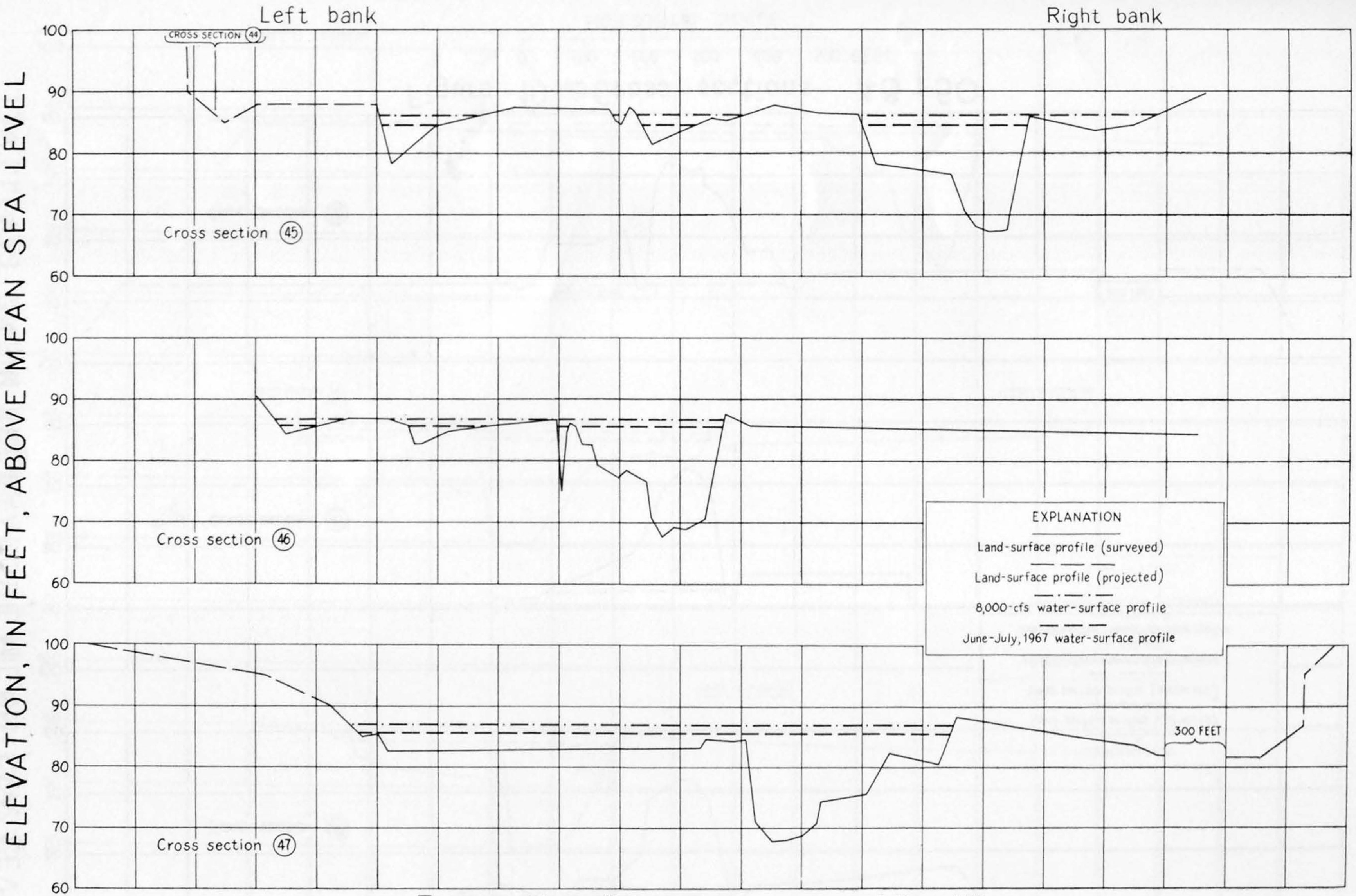


Figure 18.— Cross sections 45-47

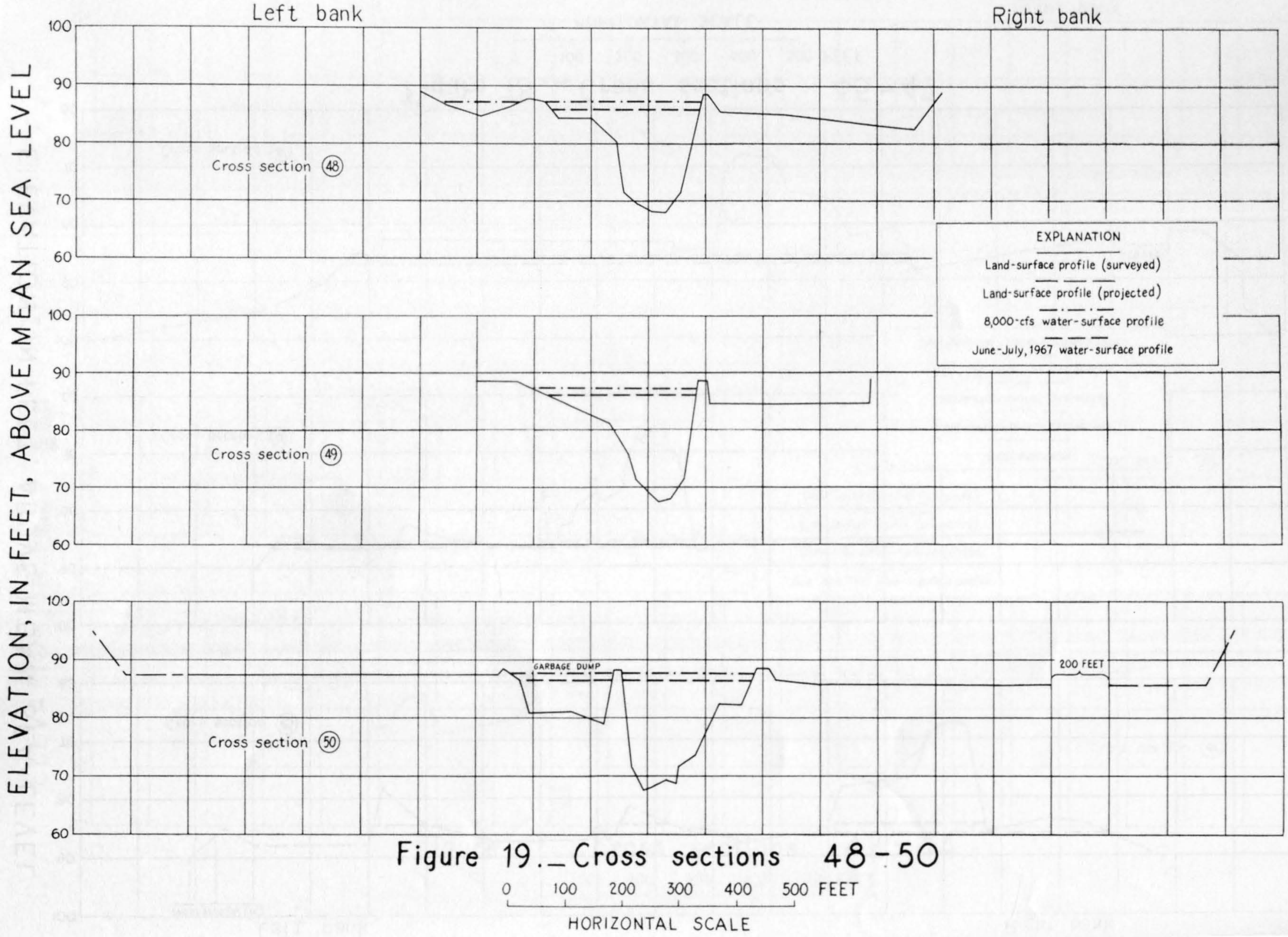


Figure 19.—Cross sections 48-50

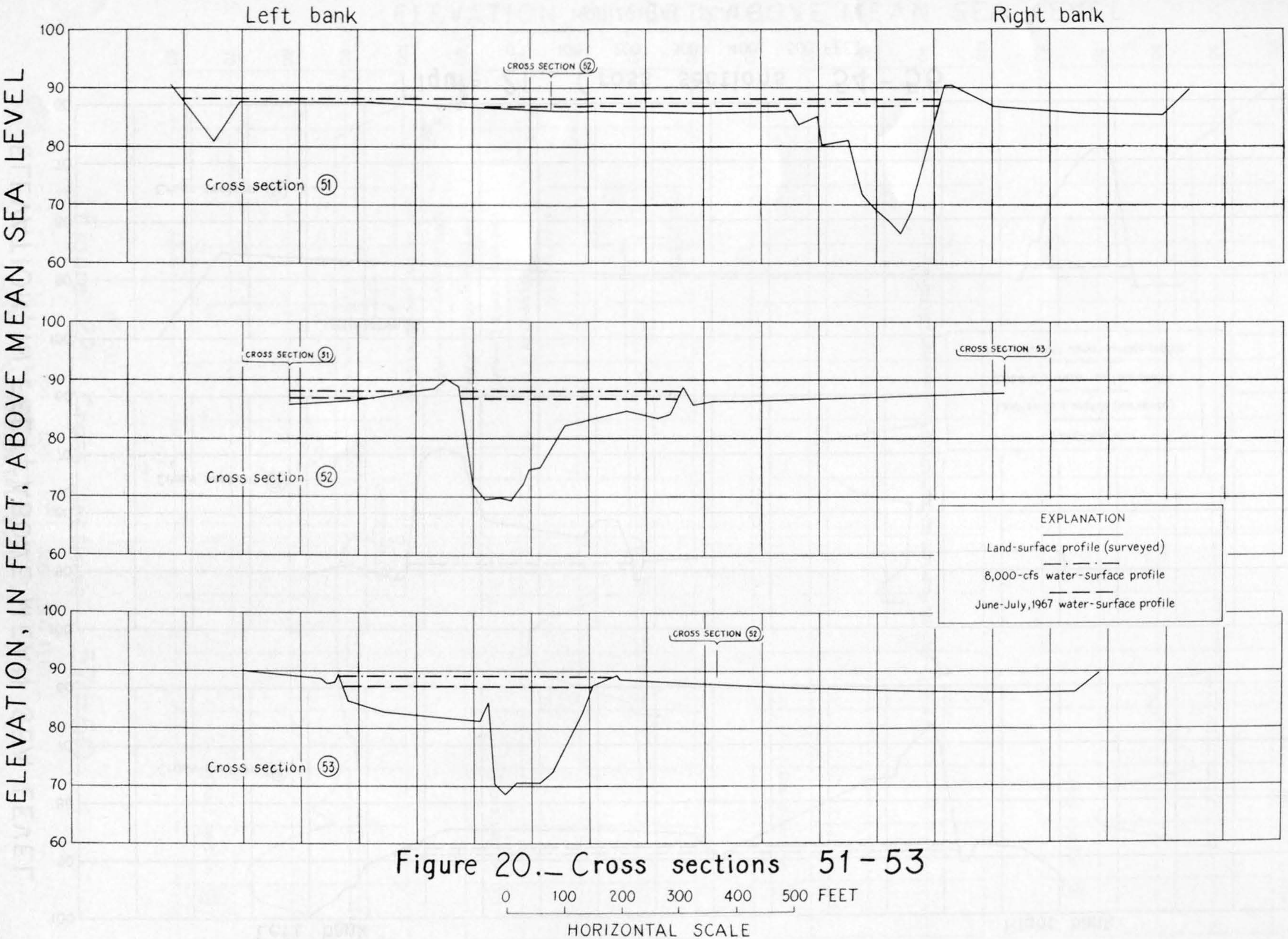


Figure 20.—Cross sections 51-53

U.S. GEOLOGICAL SURVEY

FIGURE 20

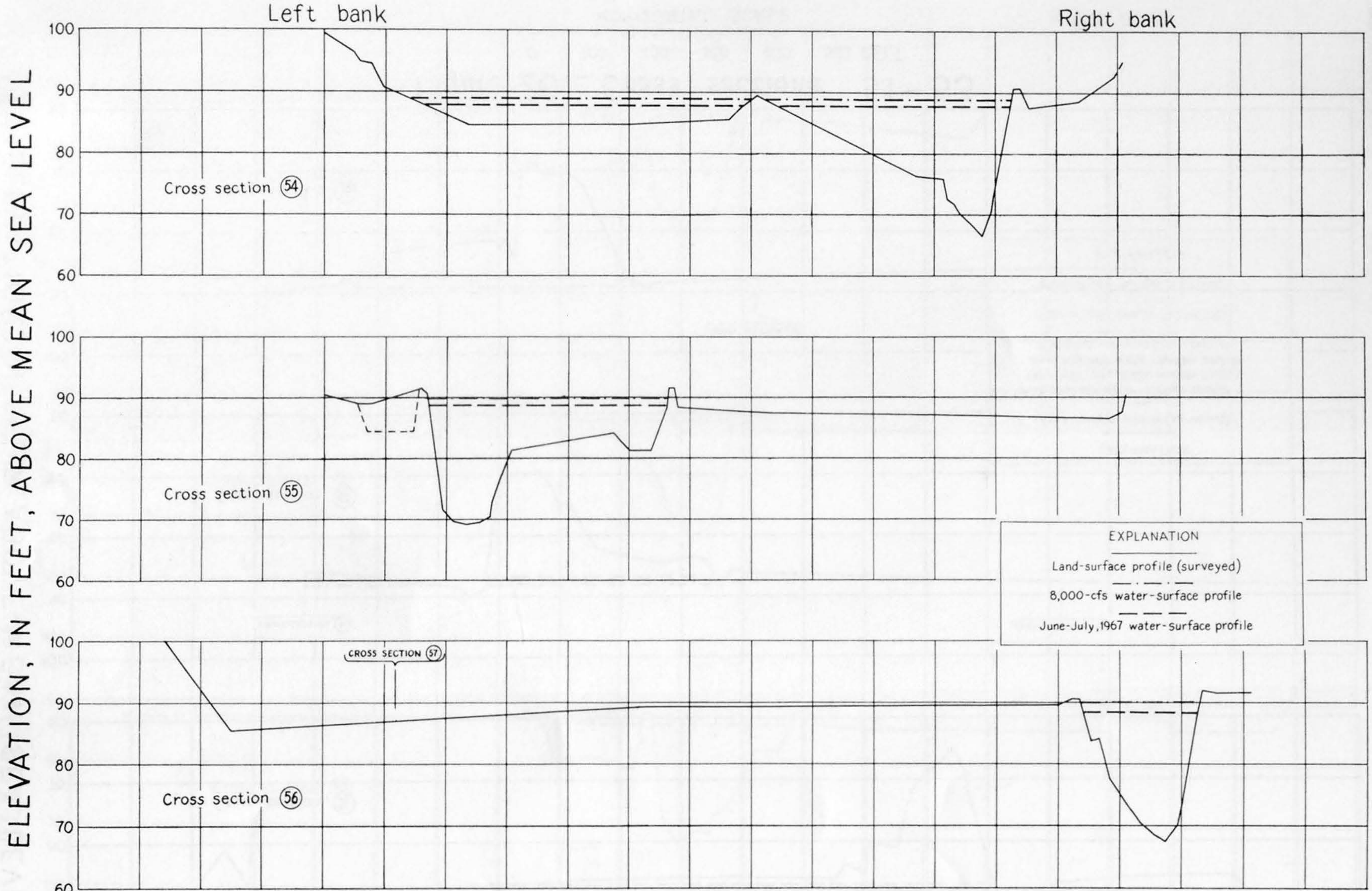


Figure 21.— Cross sections 54-56

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

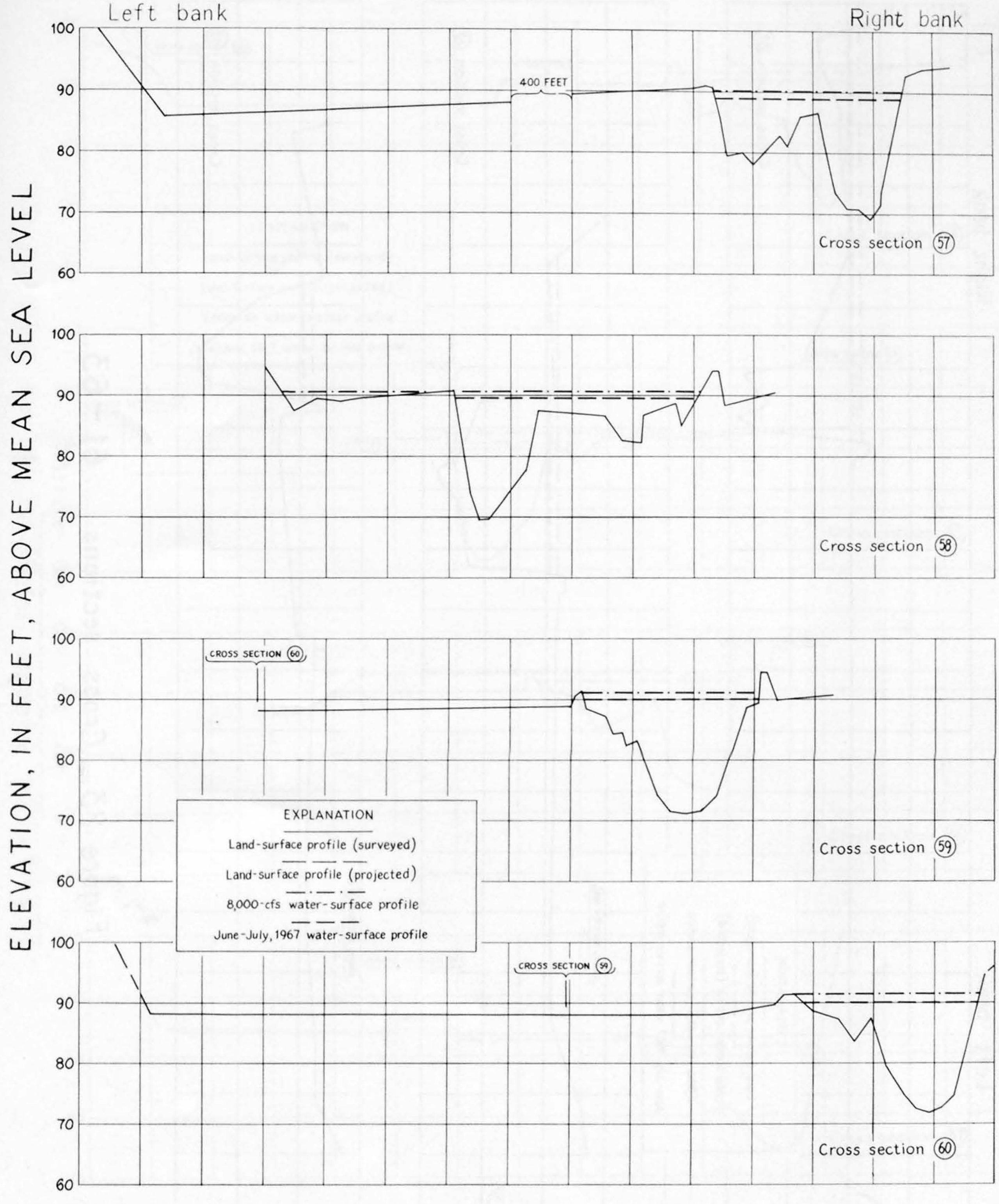


Figure 22.—Cross sections 57-60

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

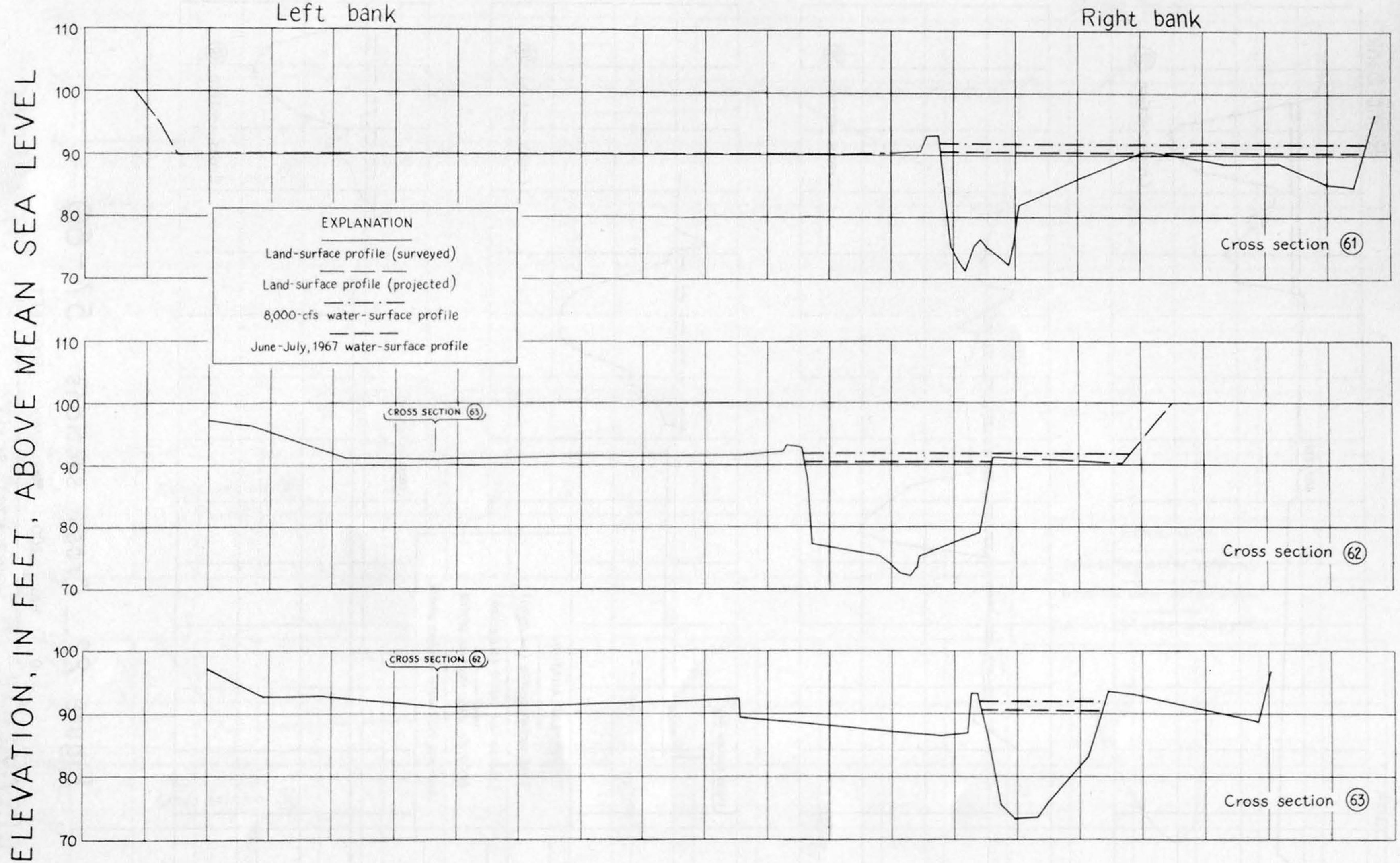


Figure 23._ Cross sections 61-63

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

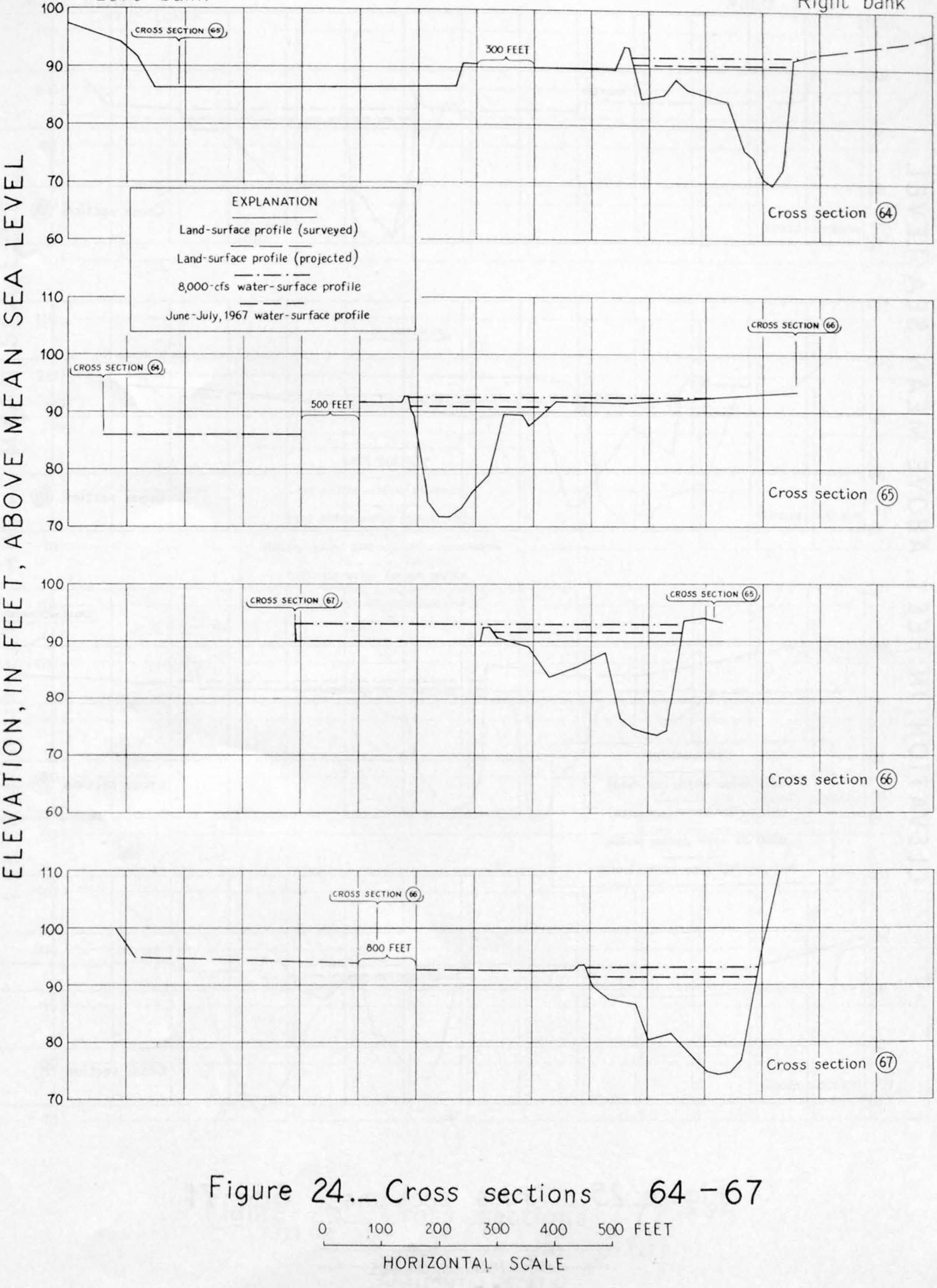


Figure 24.—Cross sections 64-67

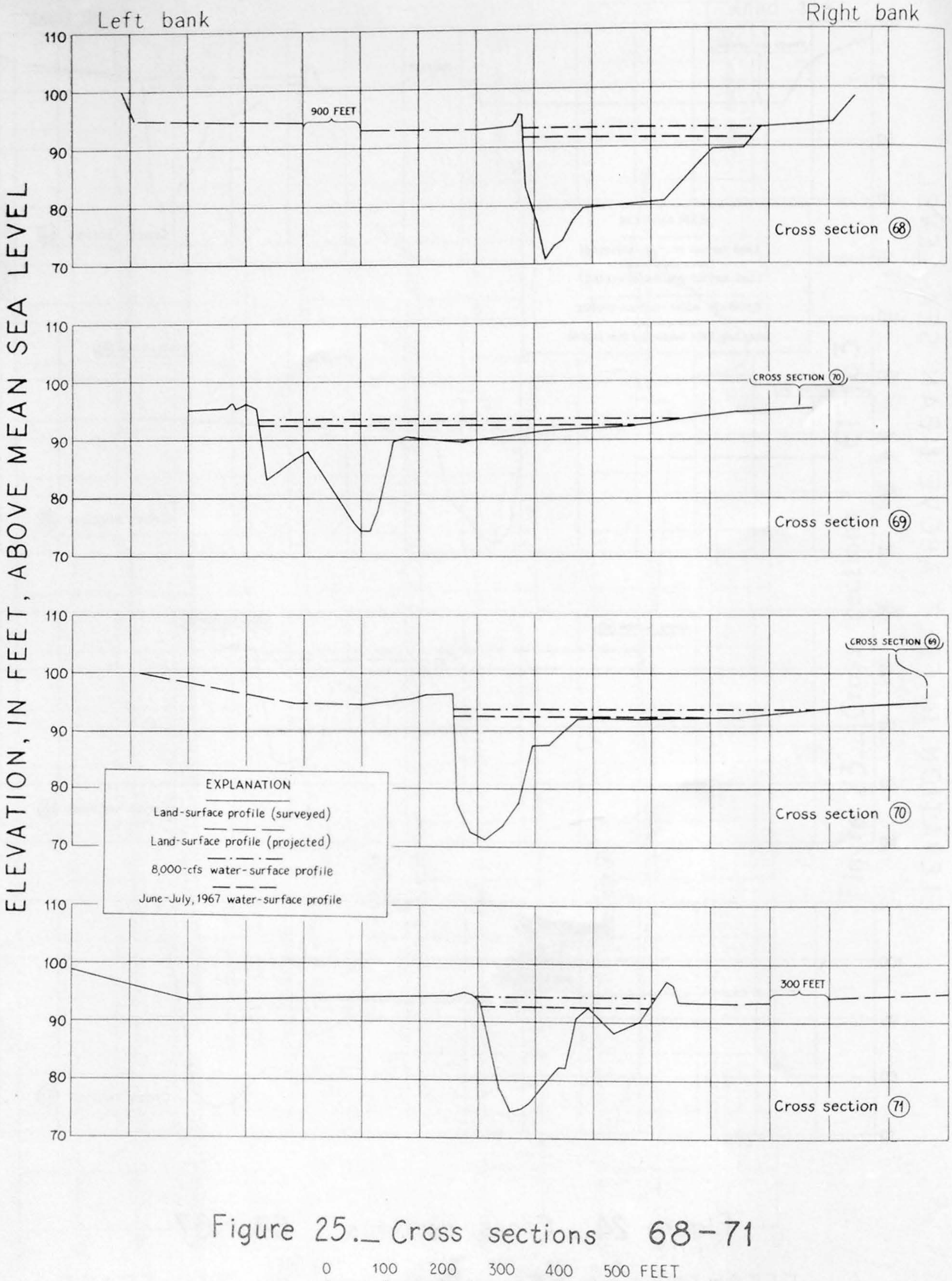


Figure 25._ Cross sections 68-71

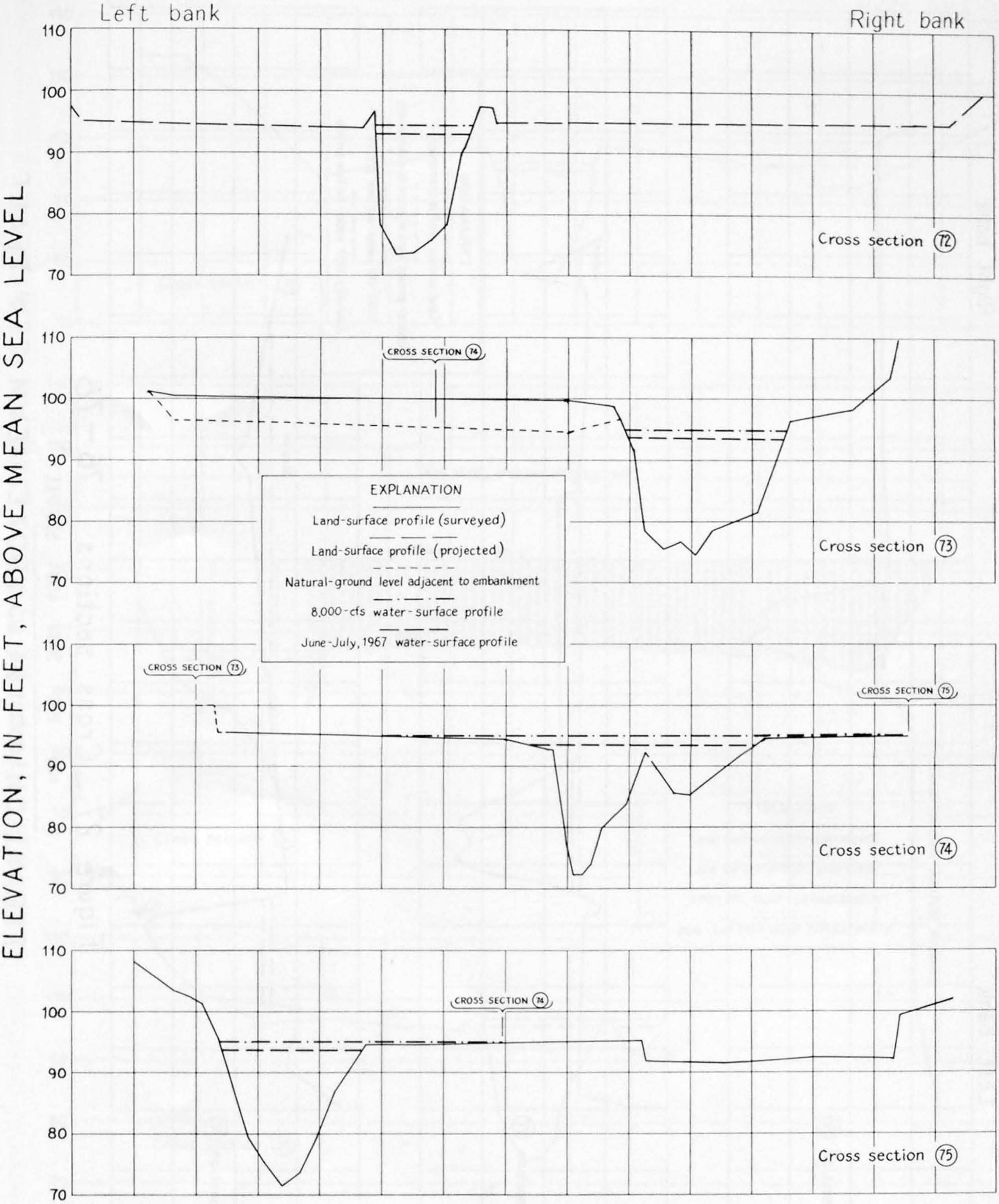


Figure 26._ Cross sections 72 – 75

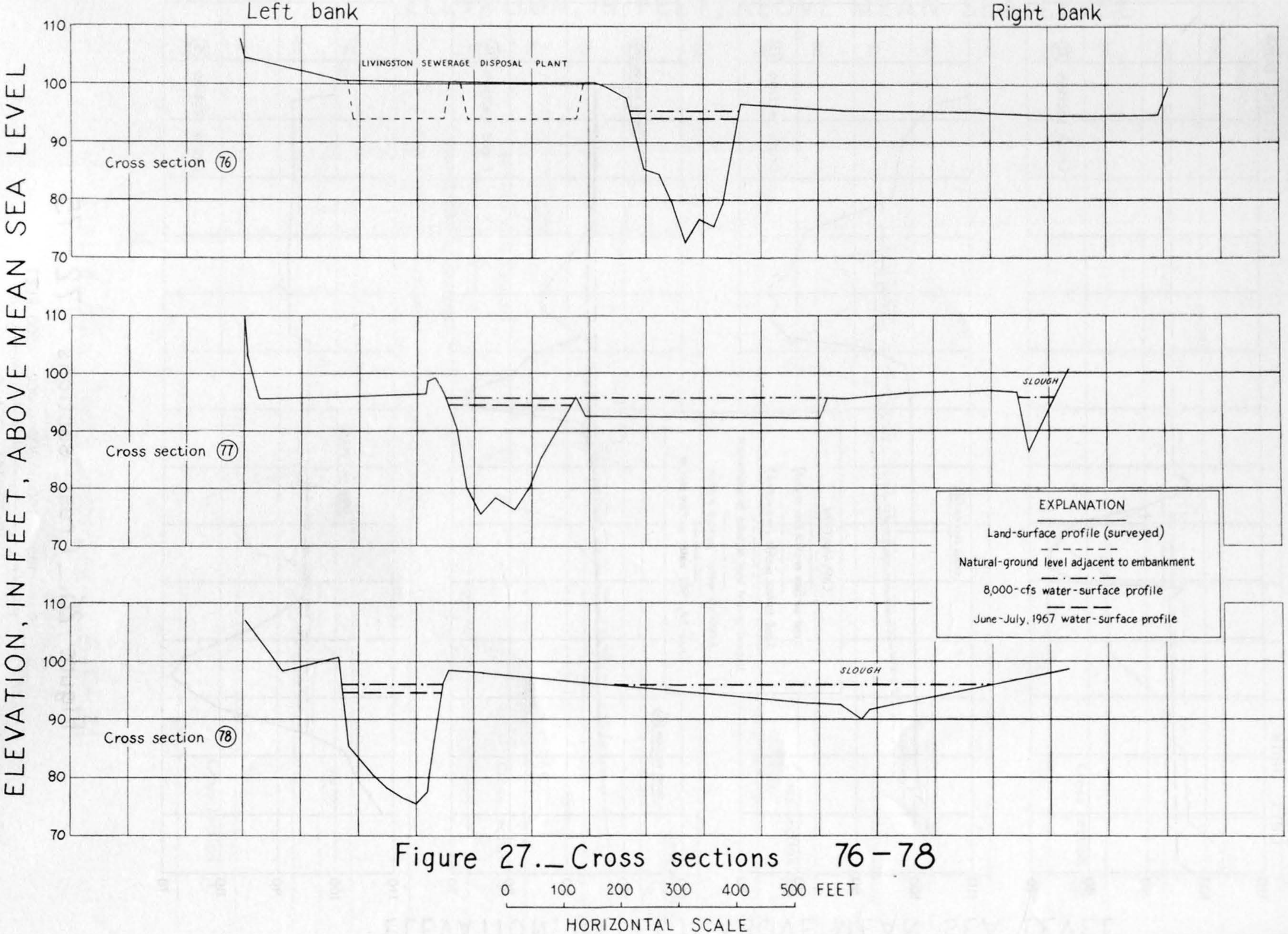


Figure 27._Cross sections 76-78

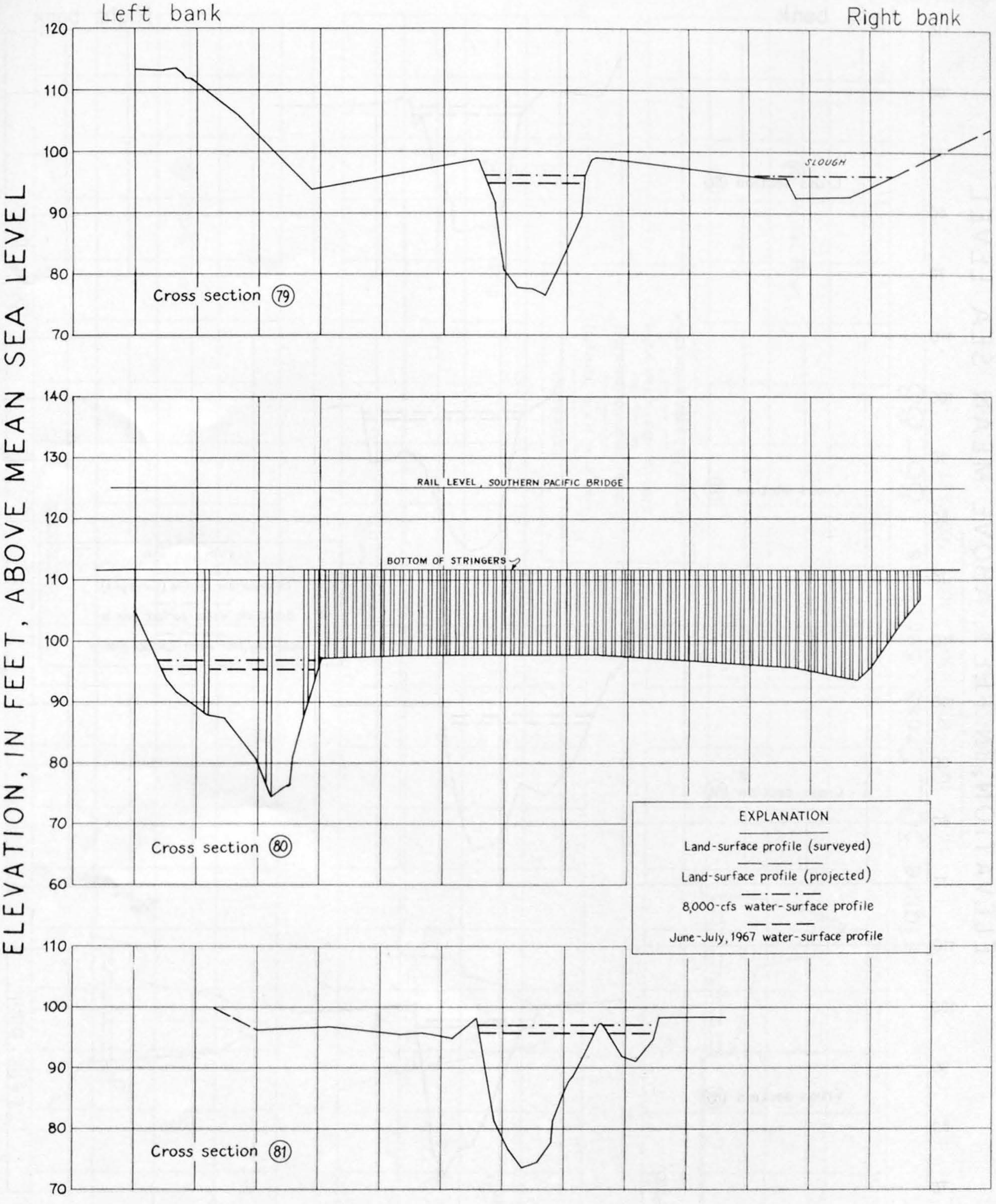


Figure 28._Cross sections 79-81

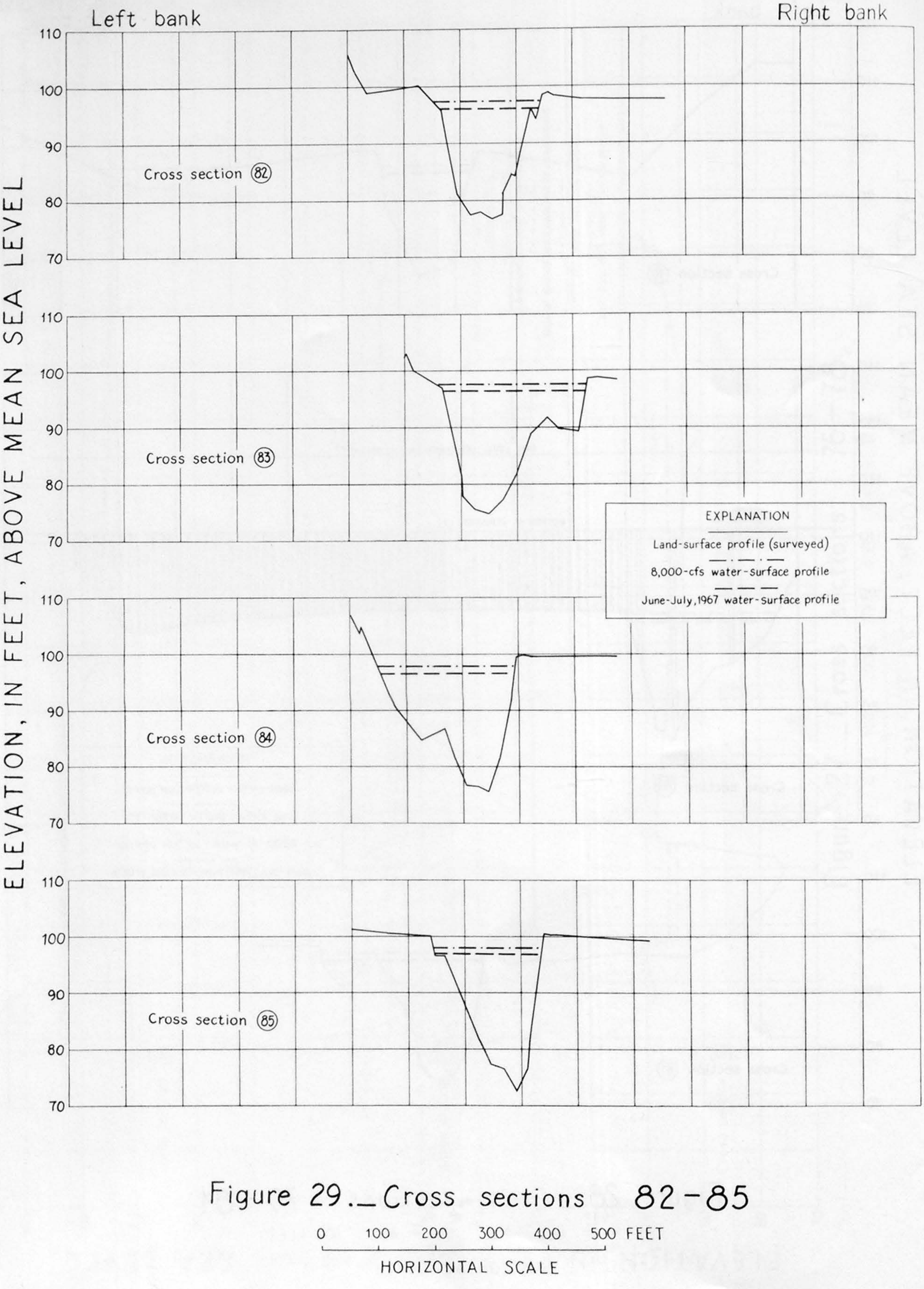


Figure 29.—Cross sections 82-85

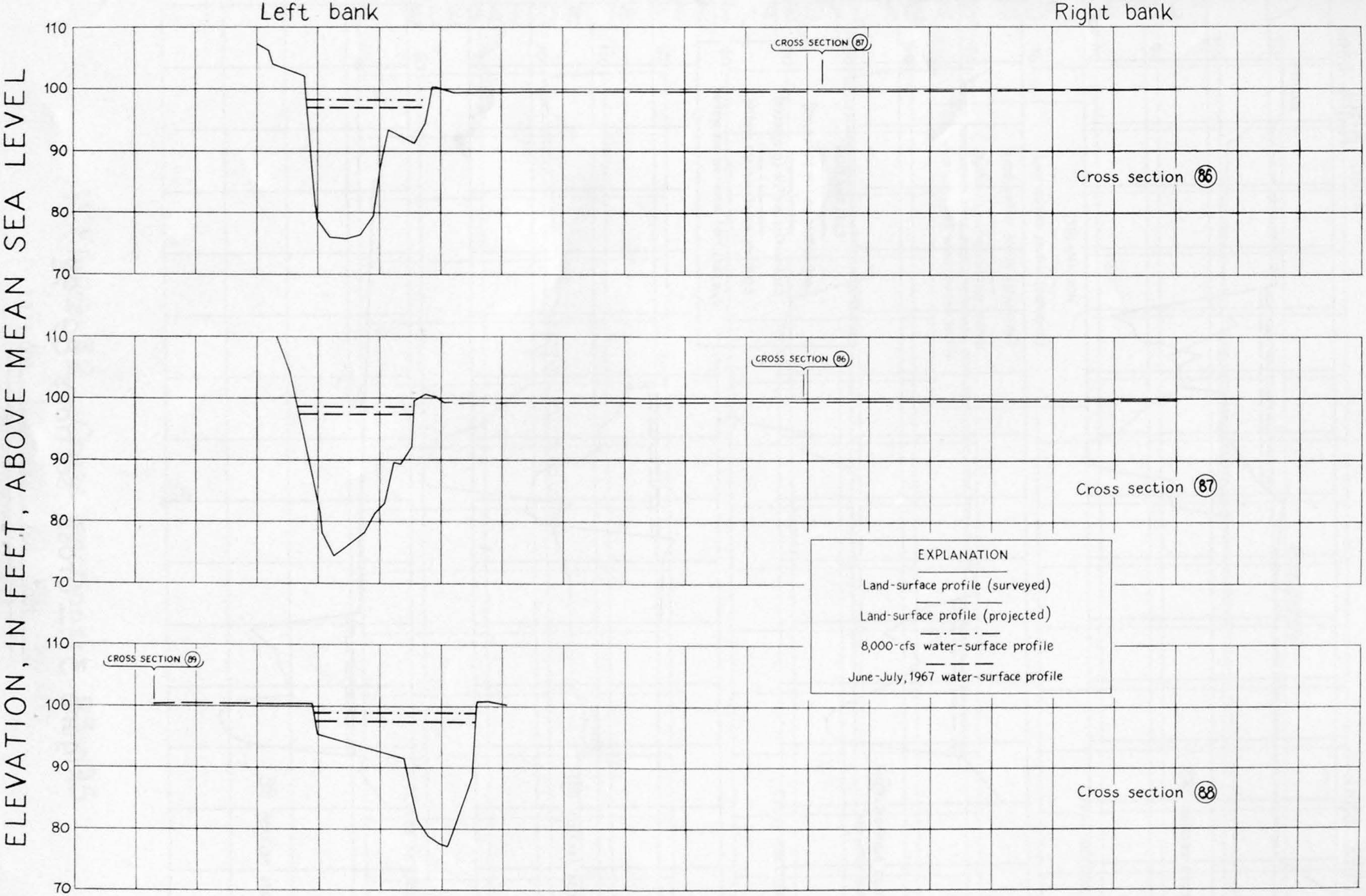


Figure 30.— Cross sections 86-88

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

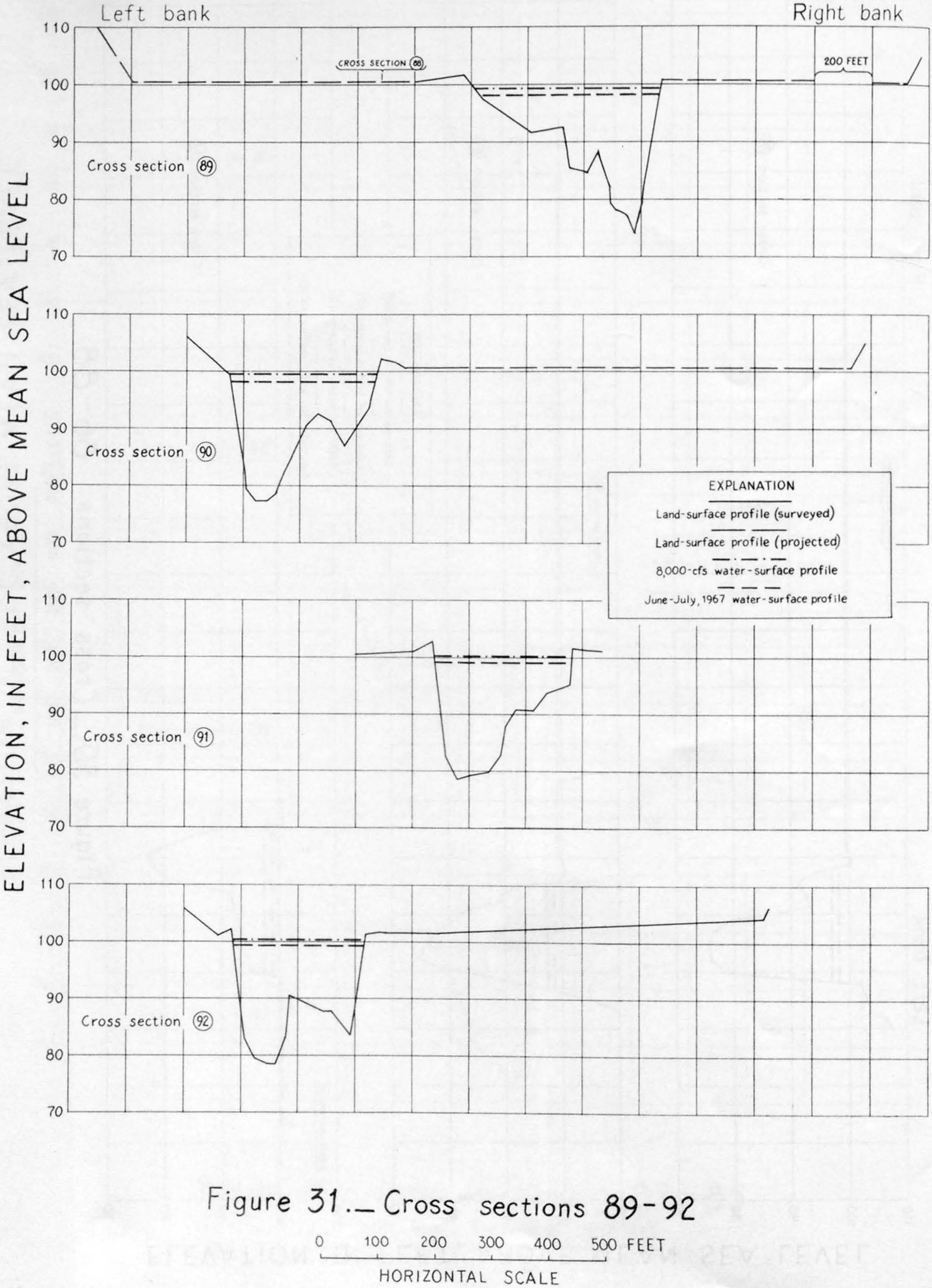


Figure 31.—Cross sections 89-92

U.S. GEOLOGICAL SURVEY

FIGURE 32

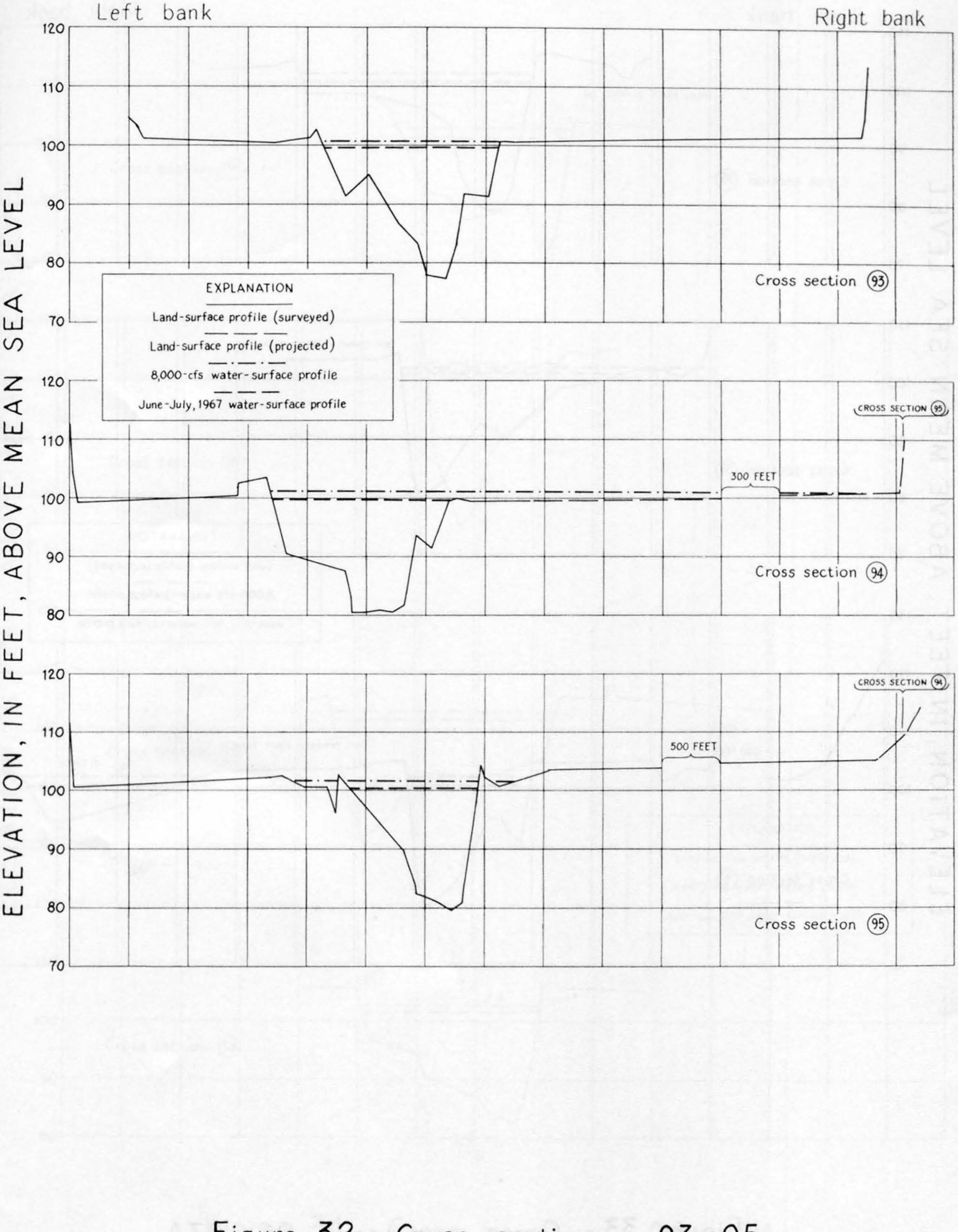


Figure 32.—Cross sections 93-95

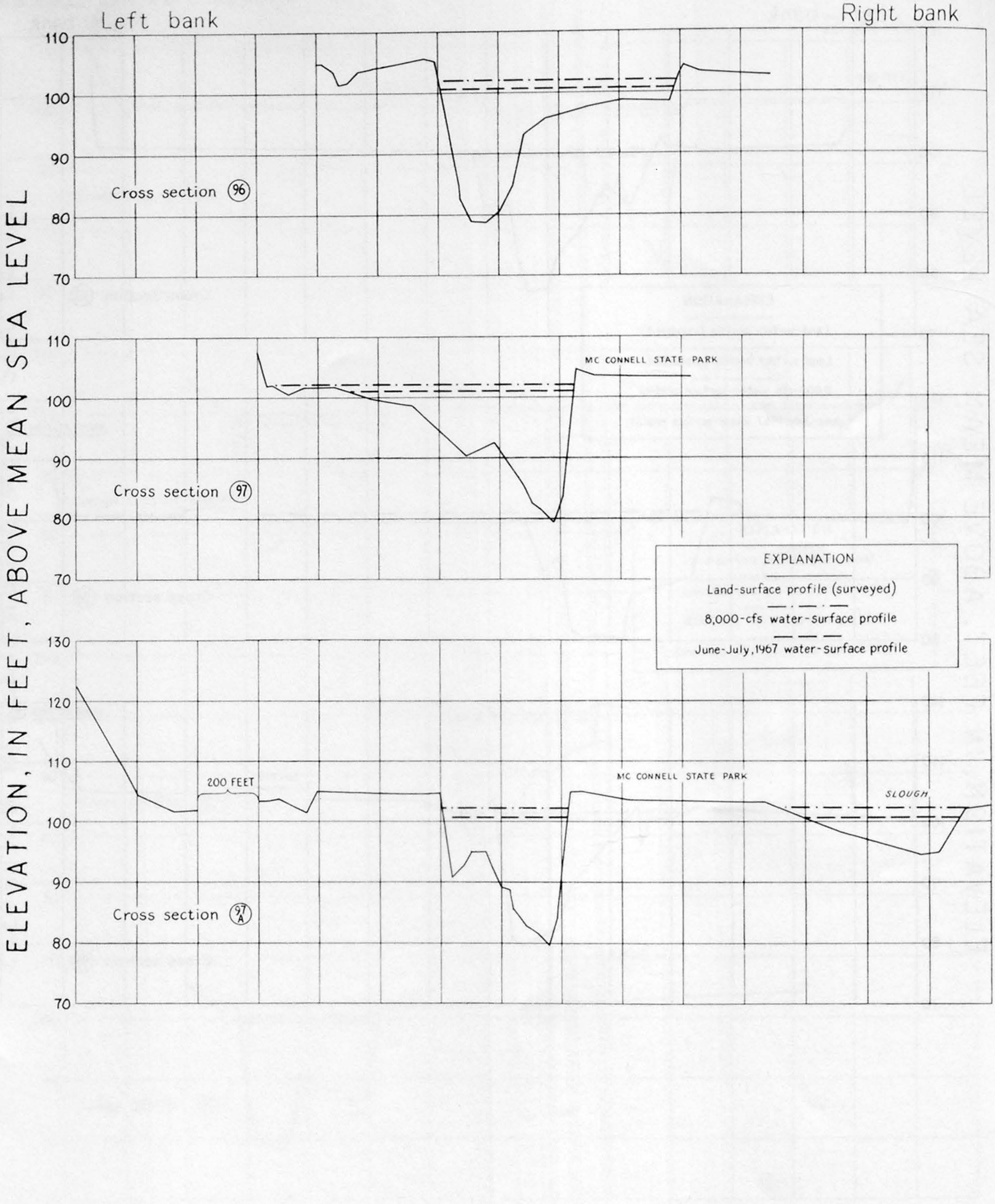


Figure 33.—Cross sections 96-97A

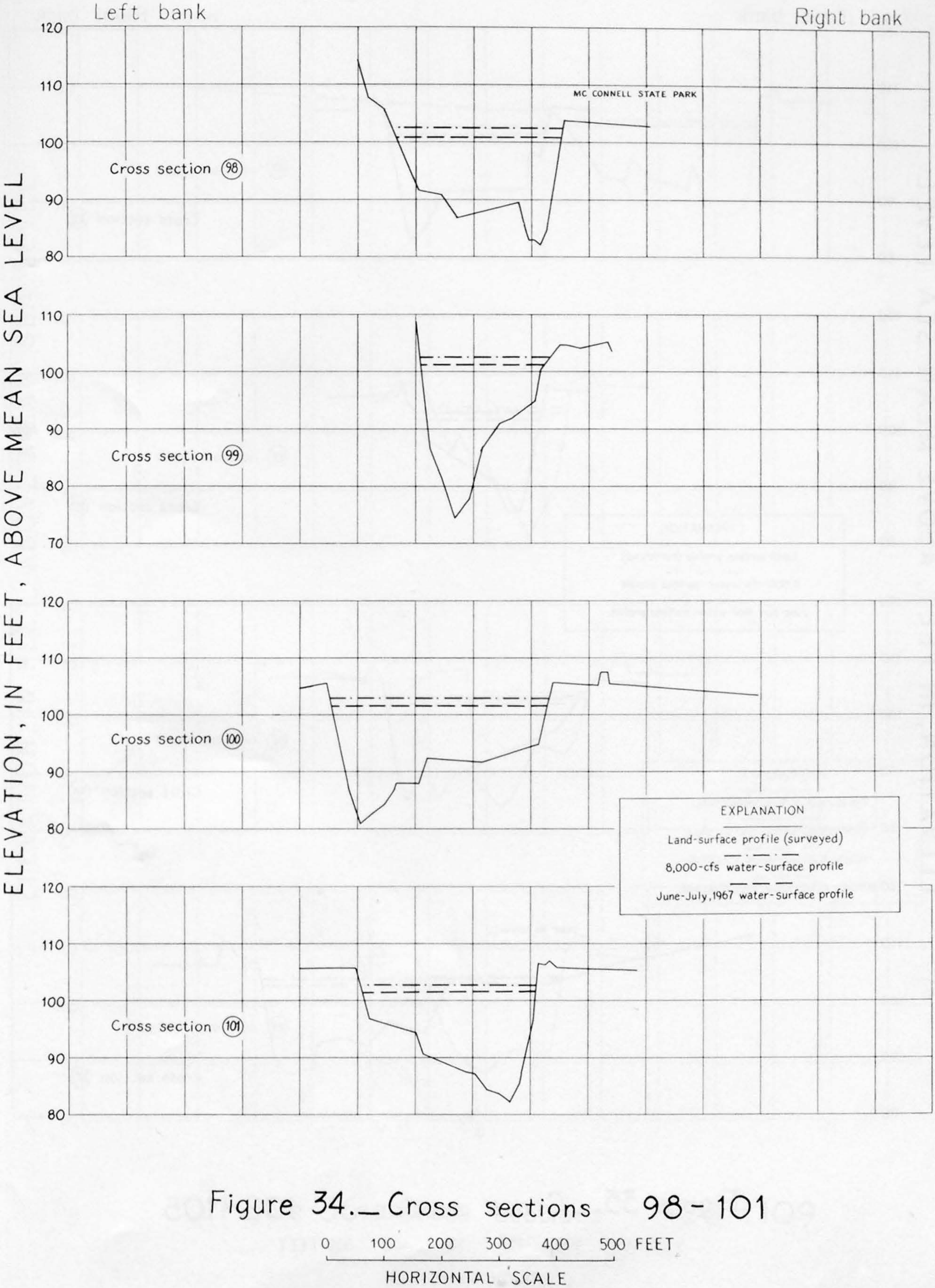


Figure 34.—Cross sections 98-101

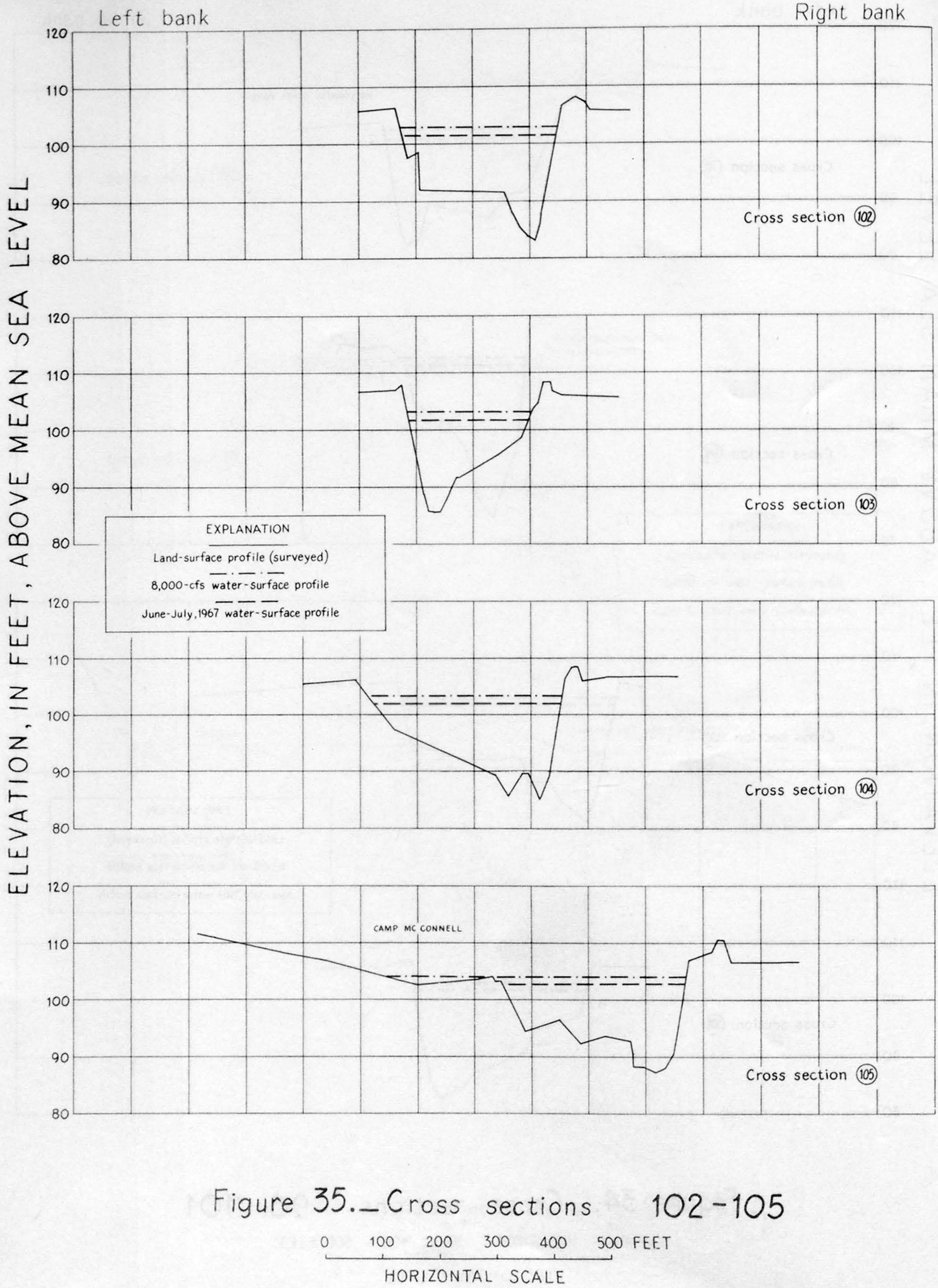


Figure 35.—Cross sections 102-105

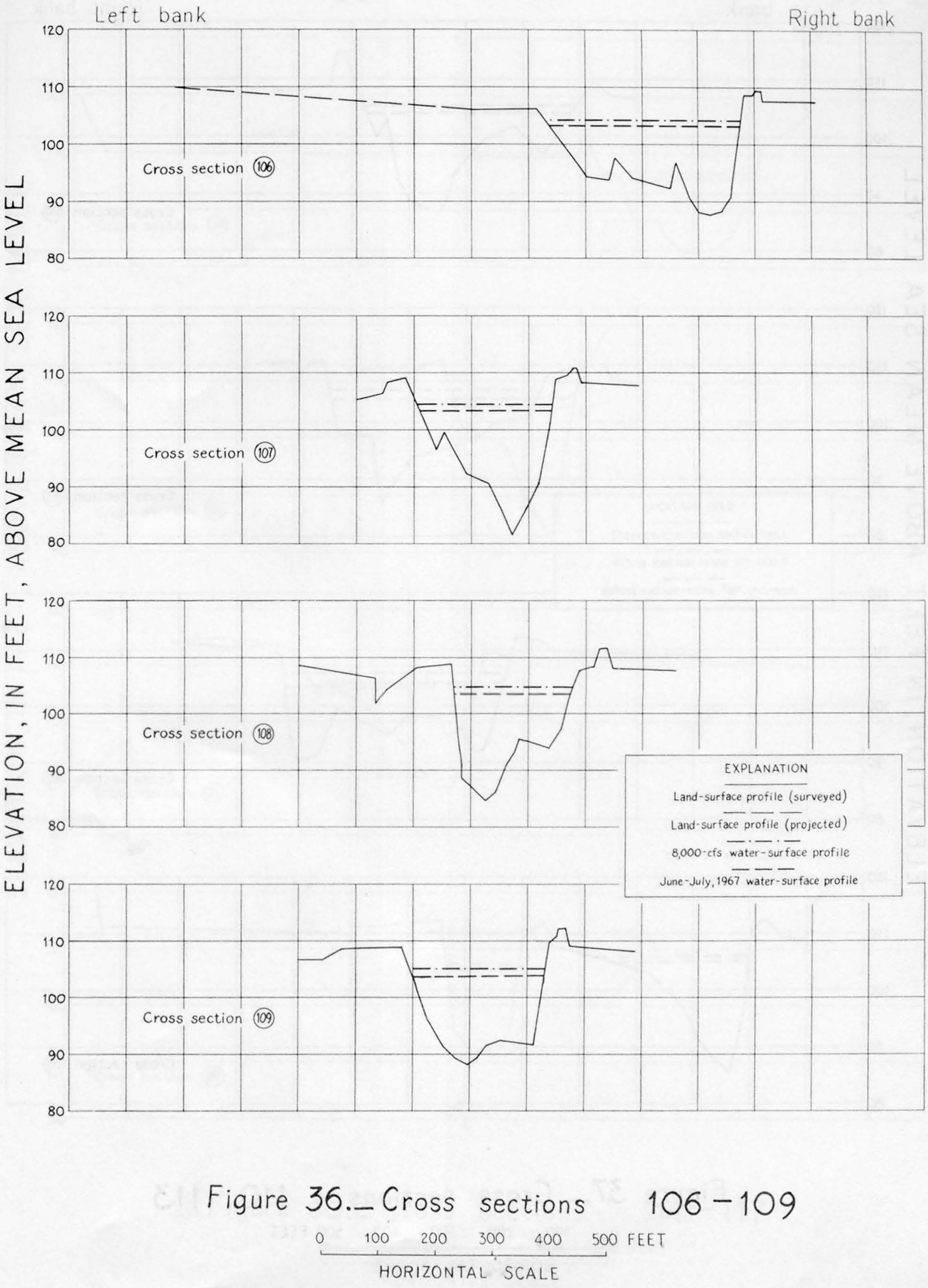


Figure 36.—Cross sections 106–109

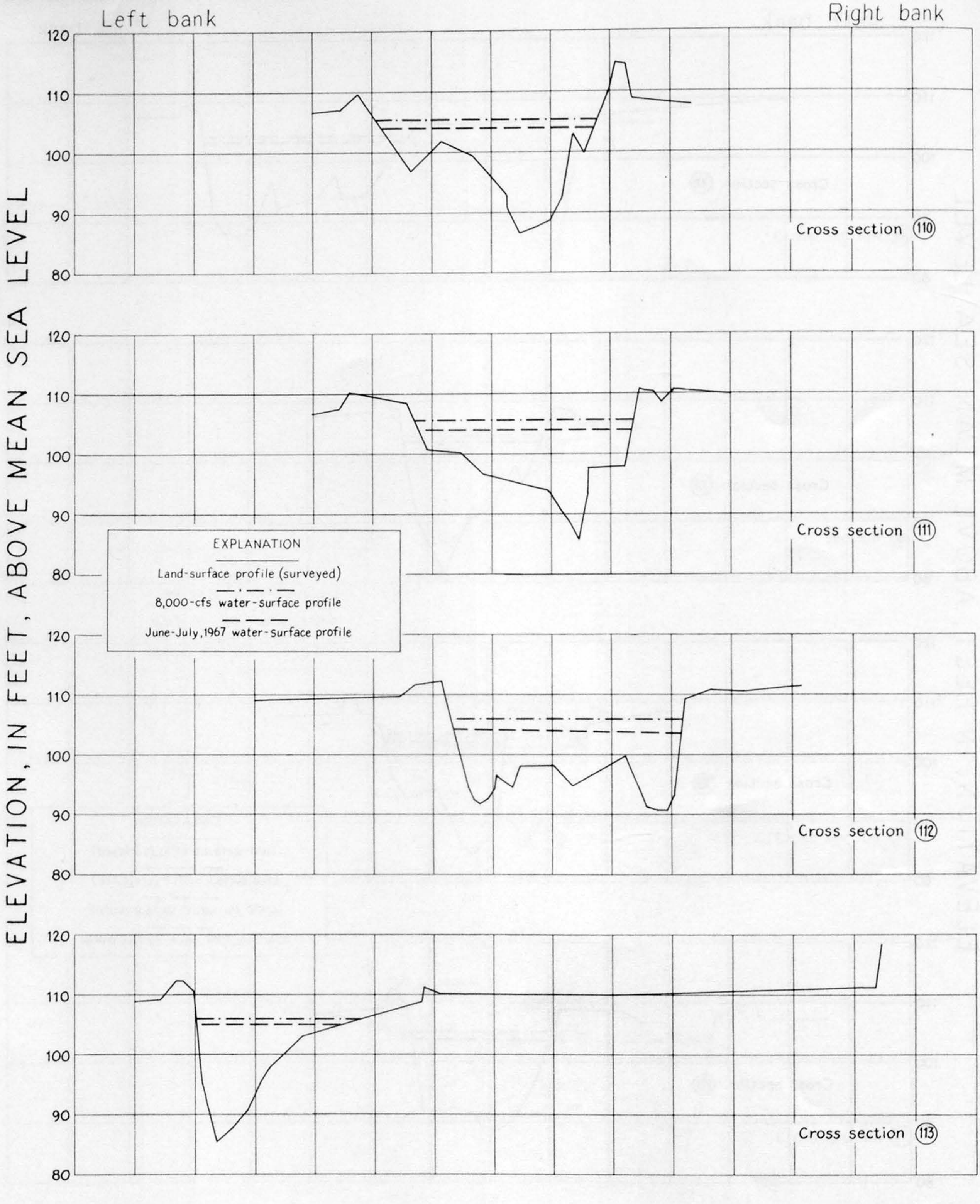


Figure 37.—Cross sections 110-113

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

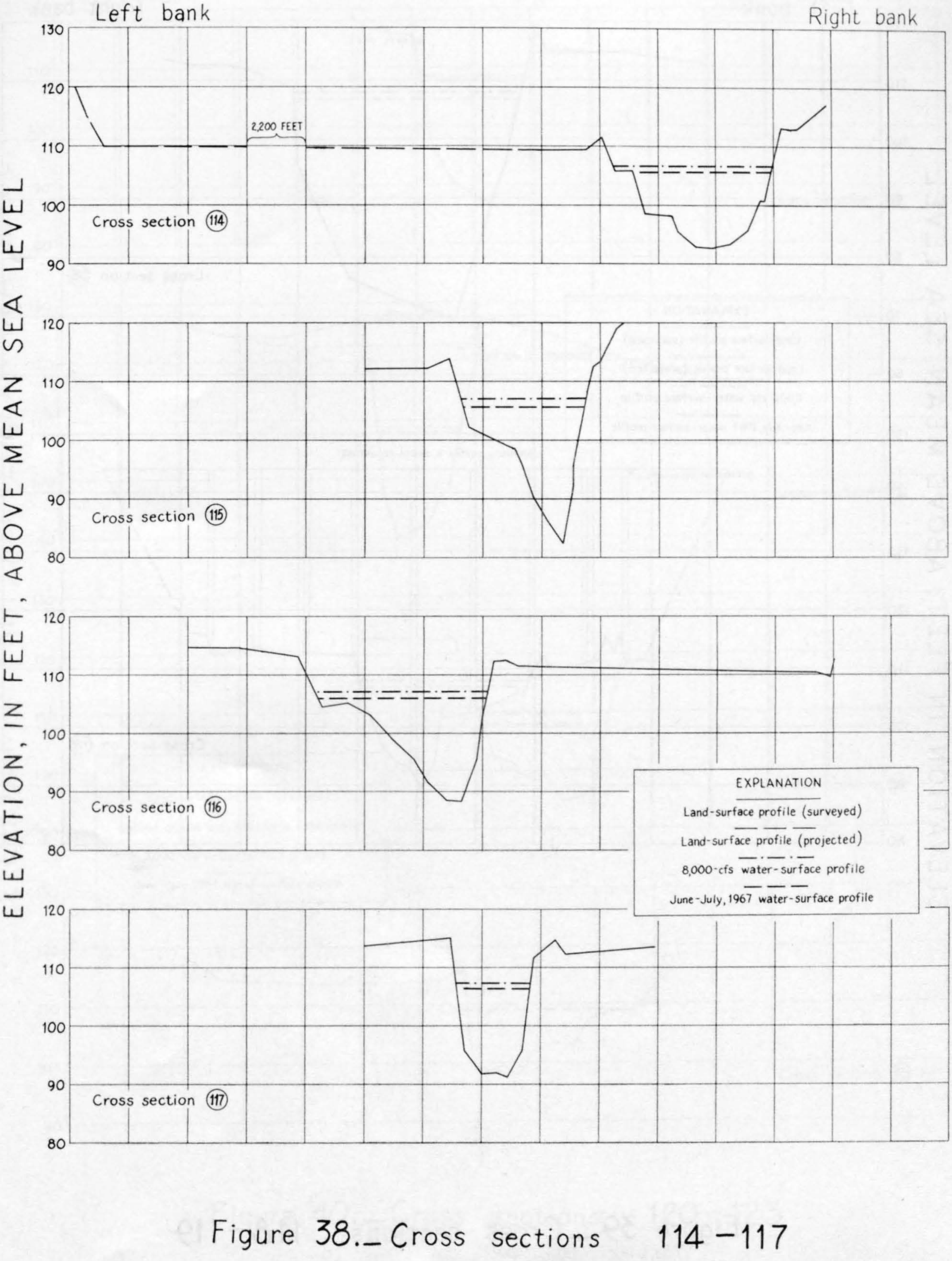


Figure 38._ Cross sections 114-117

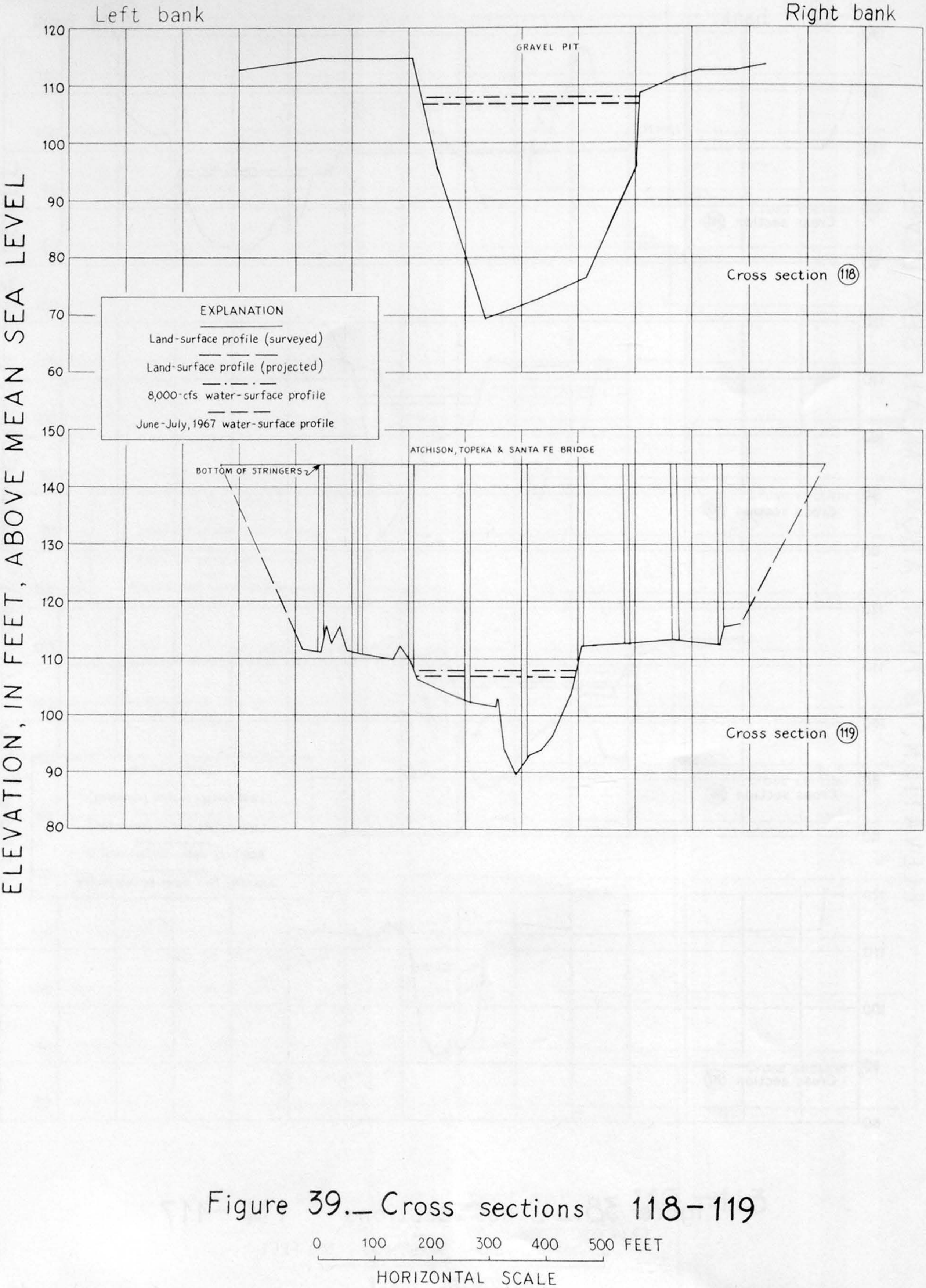


Figure 39.— Cross sections 118–119

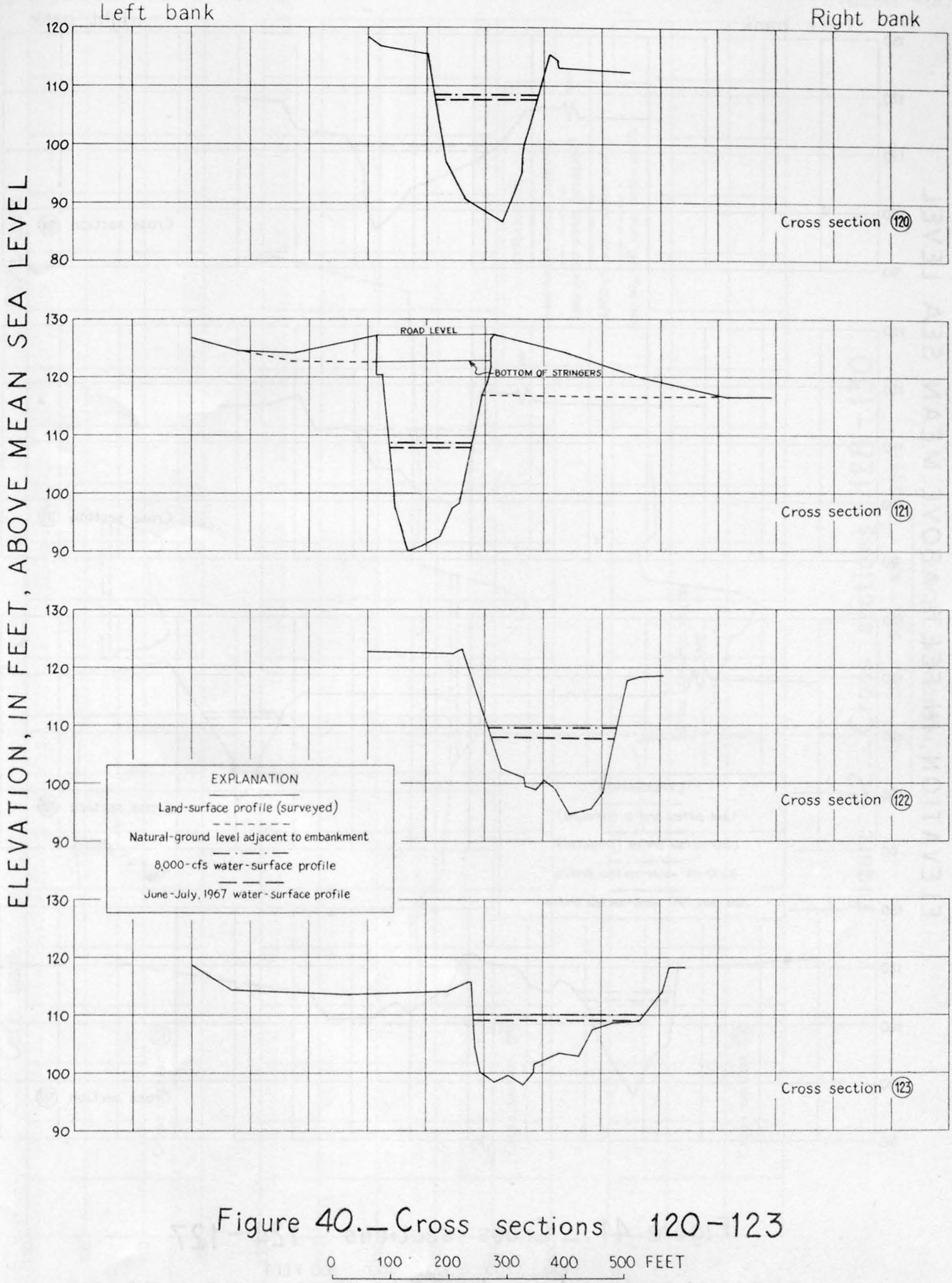


Figure 40.—Cross sections 120-123

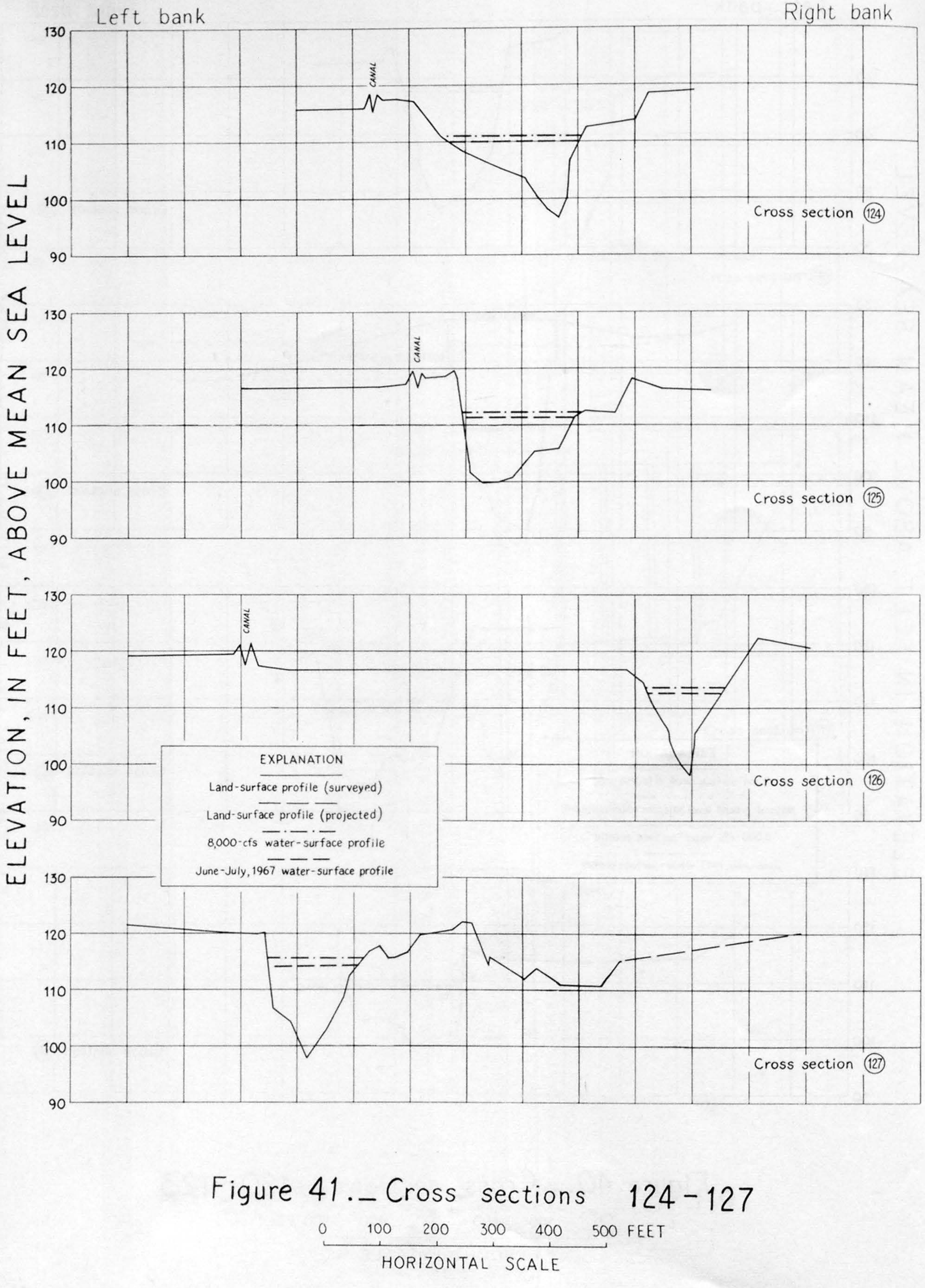


Figure 41.— Cross sections 124-127

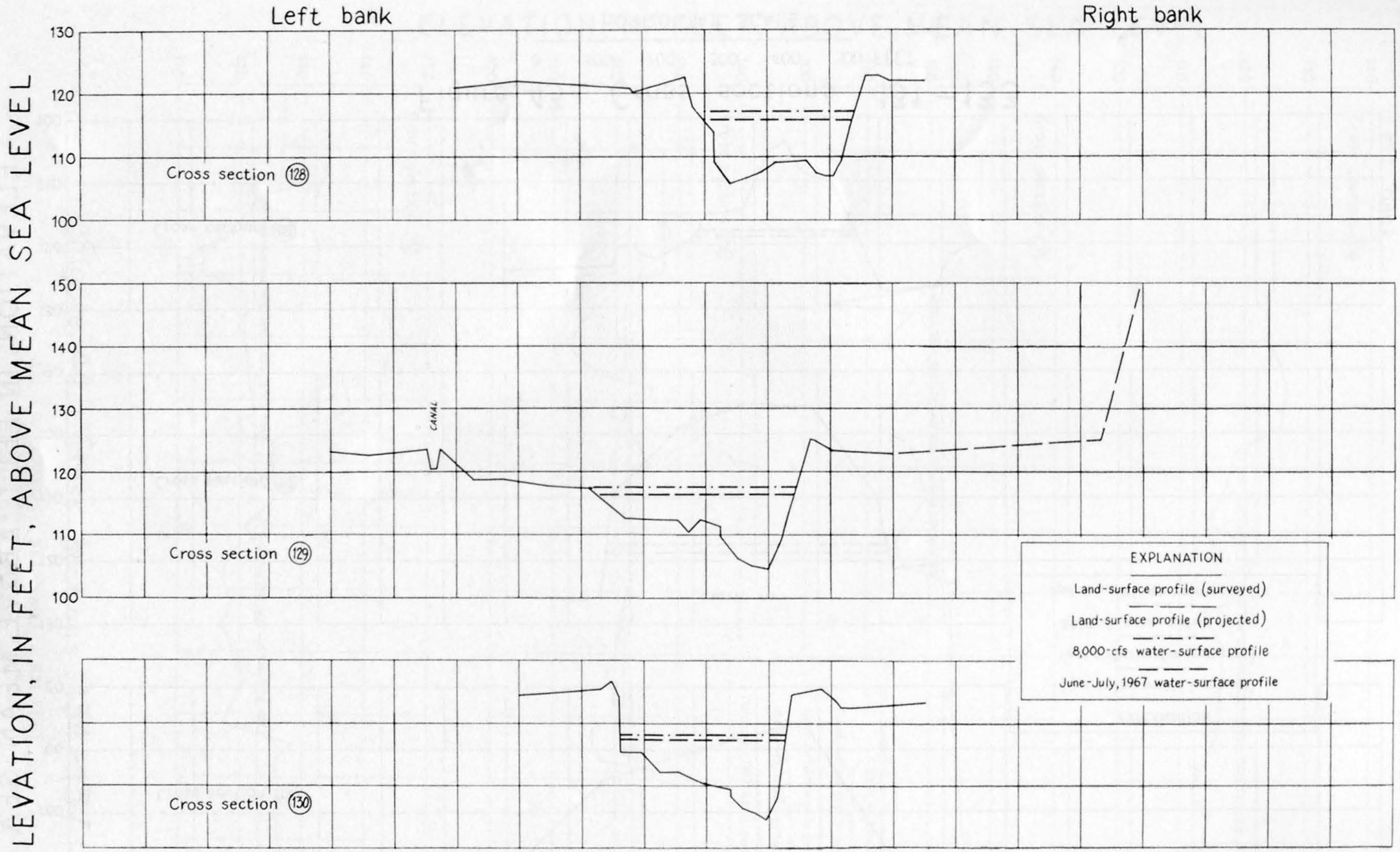


Figure 42.— Cross sections 128-130

0 100 200 300 400 500 FEET
HORIZONTAL SCALE

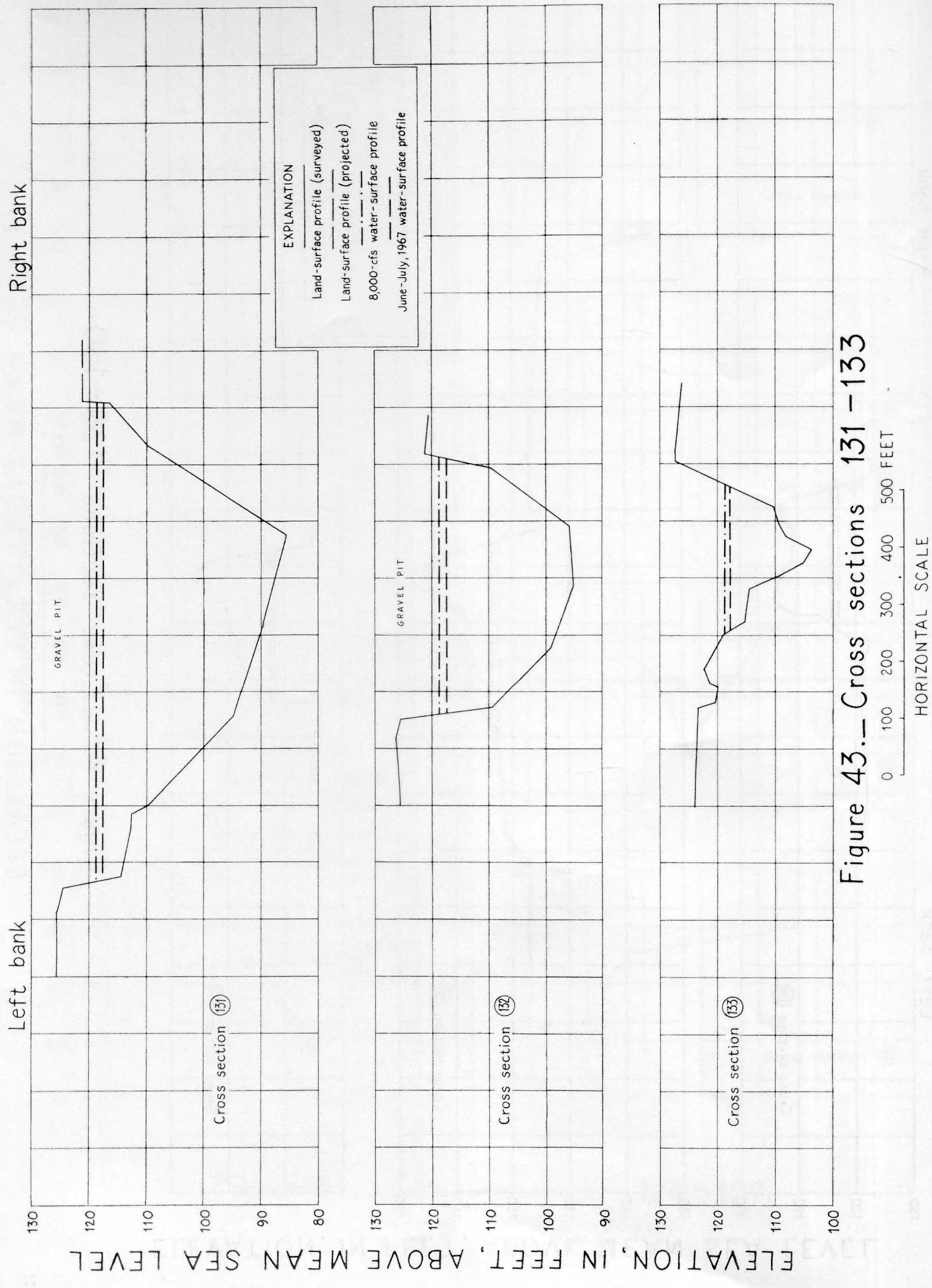


Figure 43.— Cross sections 131 – 133

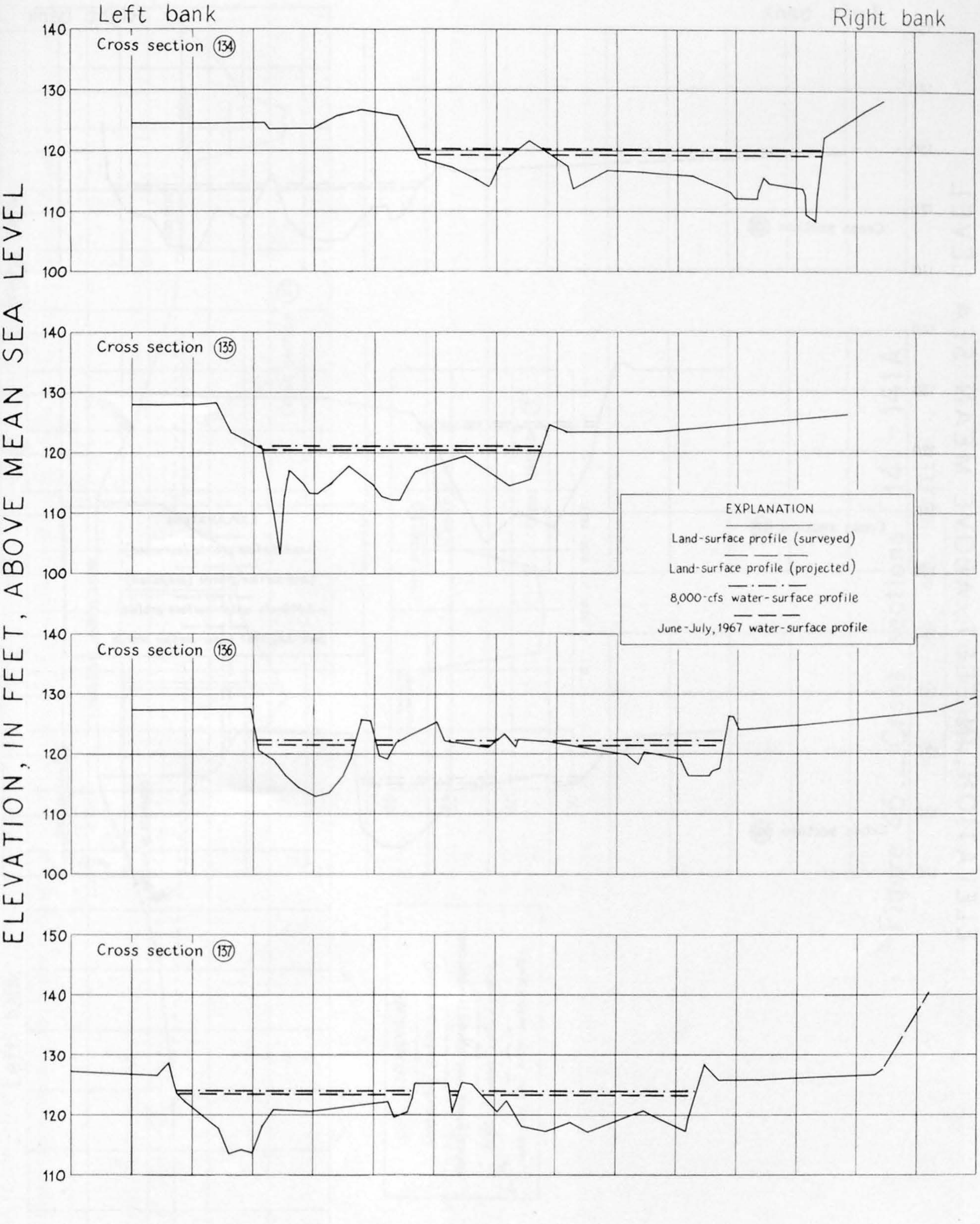


Figure 44.—Cross sections 134-137

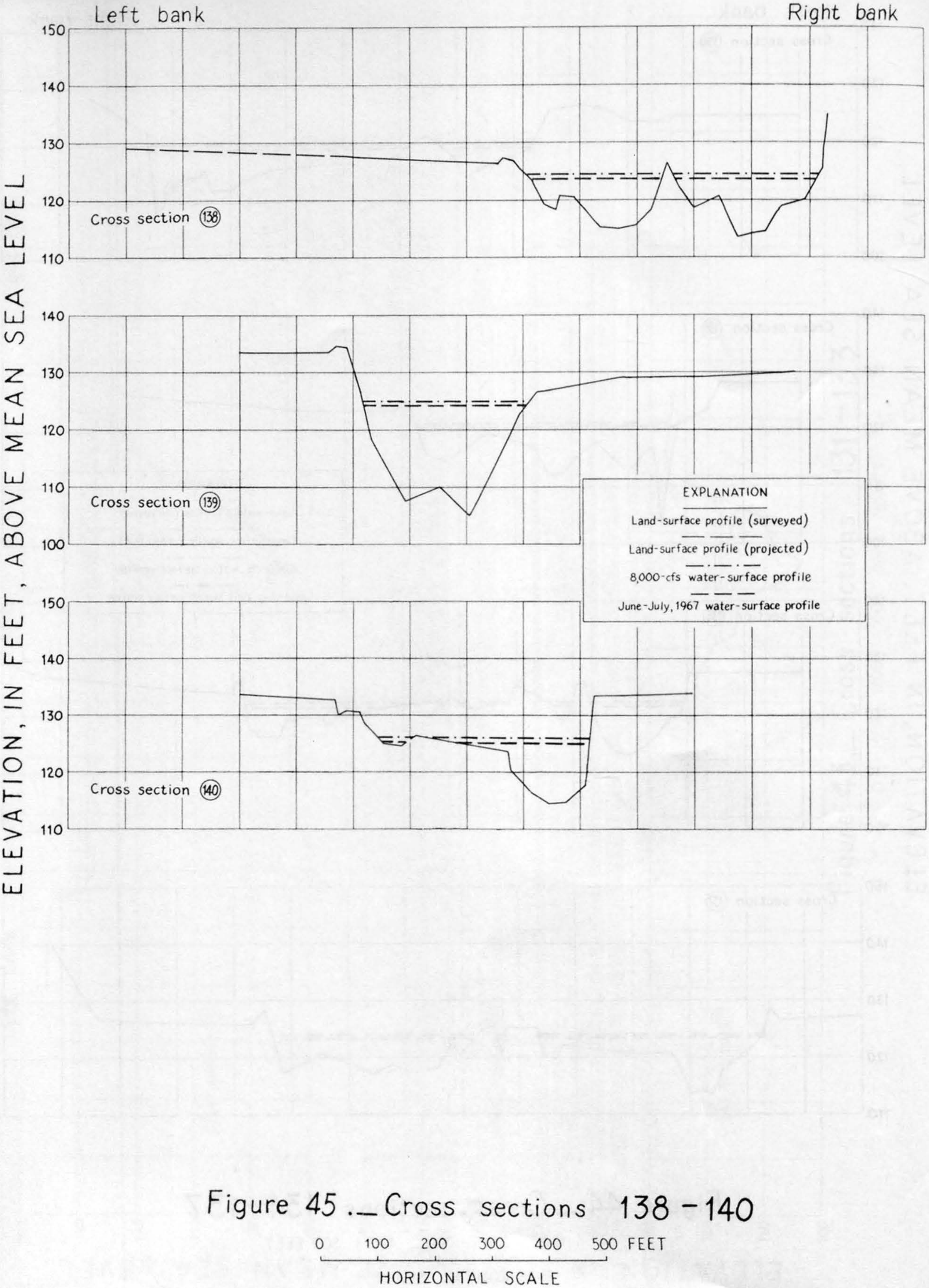


Figure 45 .— Cross sections 138 - 140

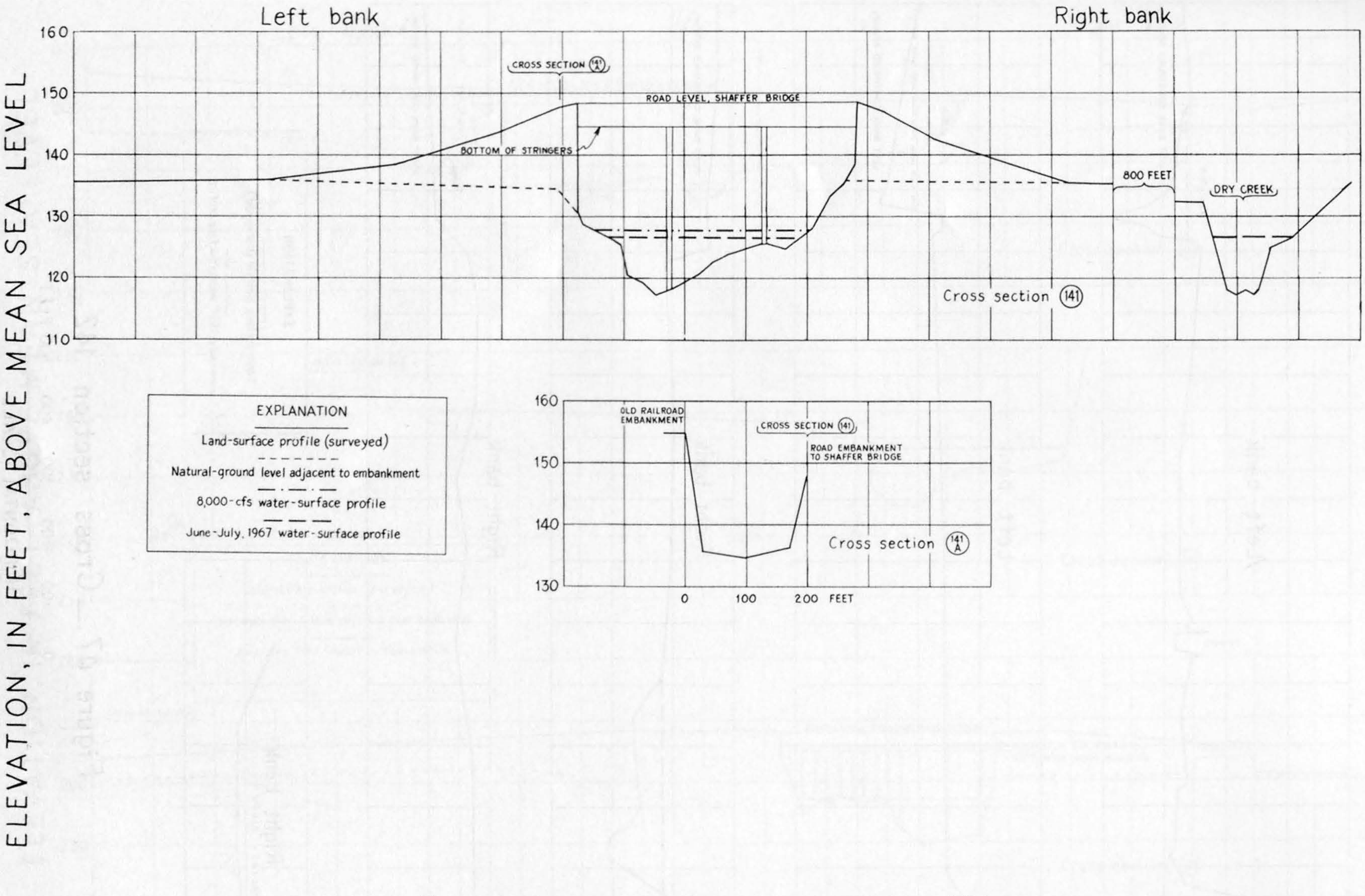


Figure 46.—Cross sections 141-141A

0 100 200 300 400 500 FEET

HORIZONTAL SCALE

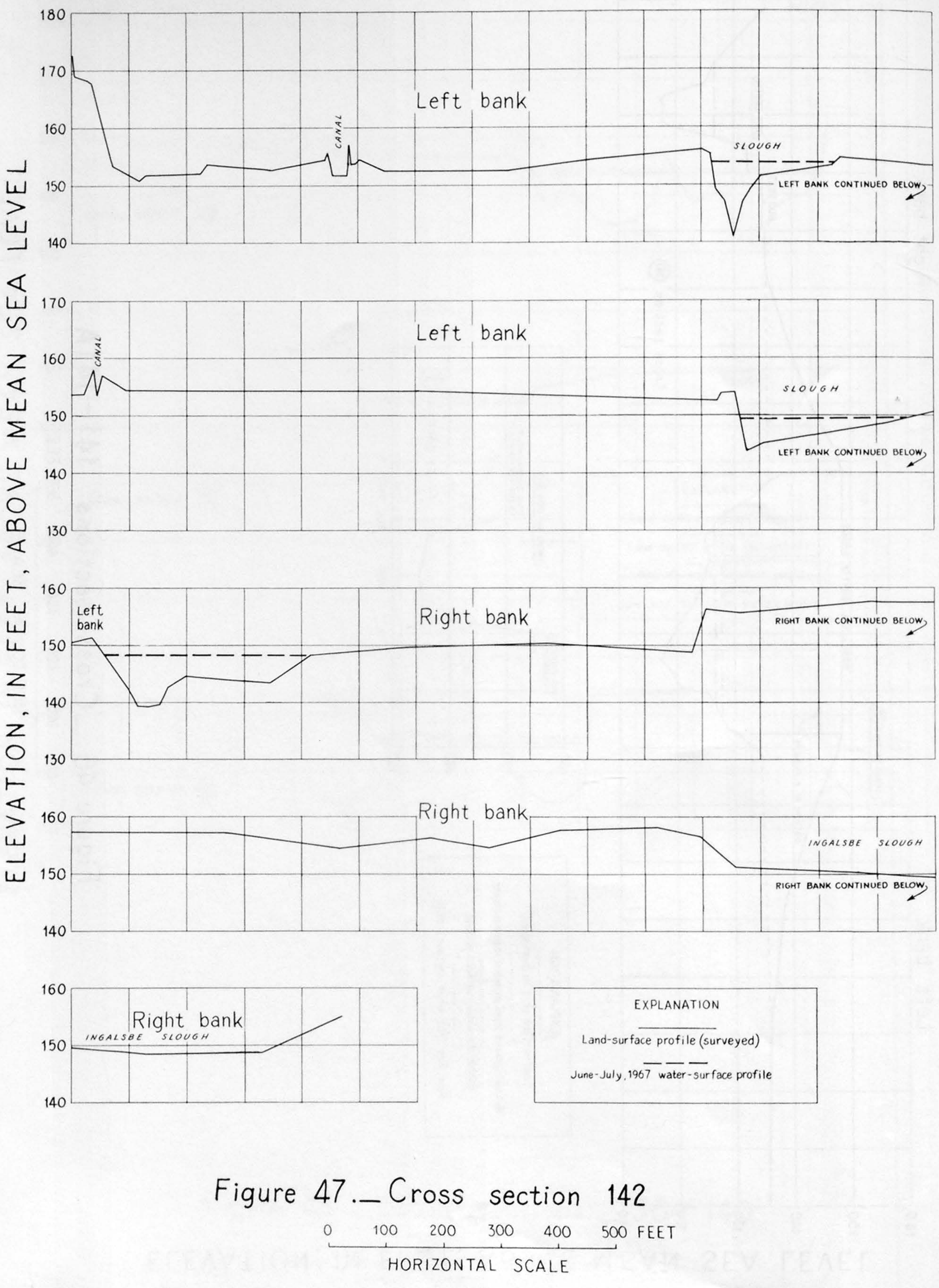


Figure 47.—Cross section 142

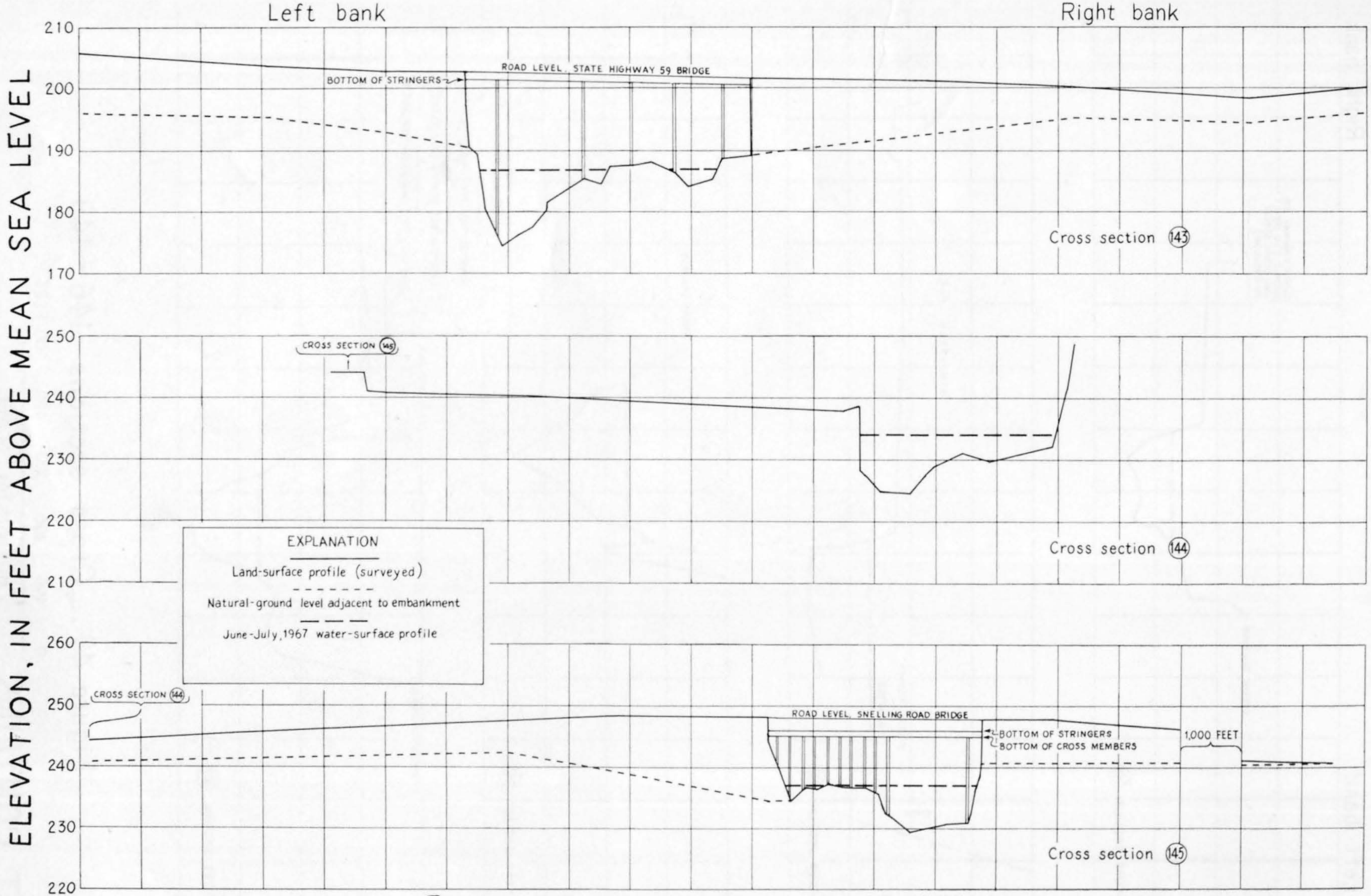


Figure 48.—Cross sections 143-145

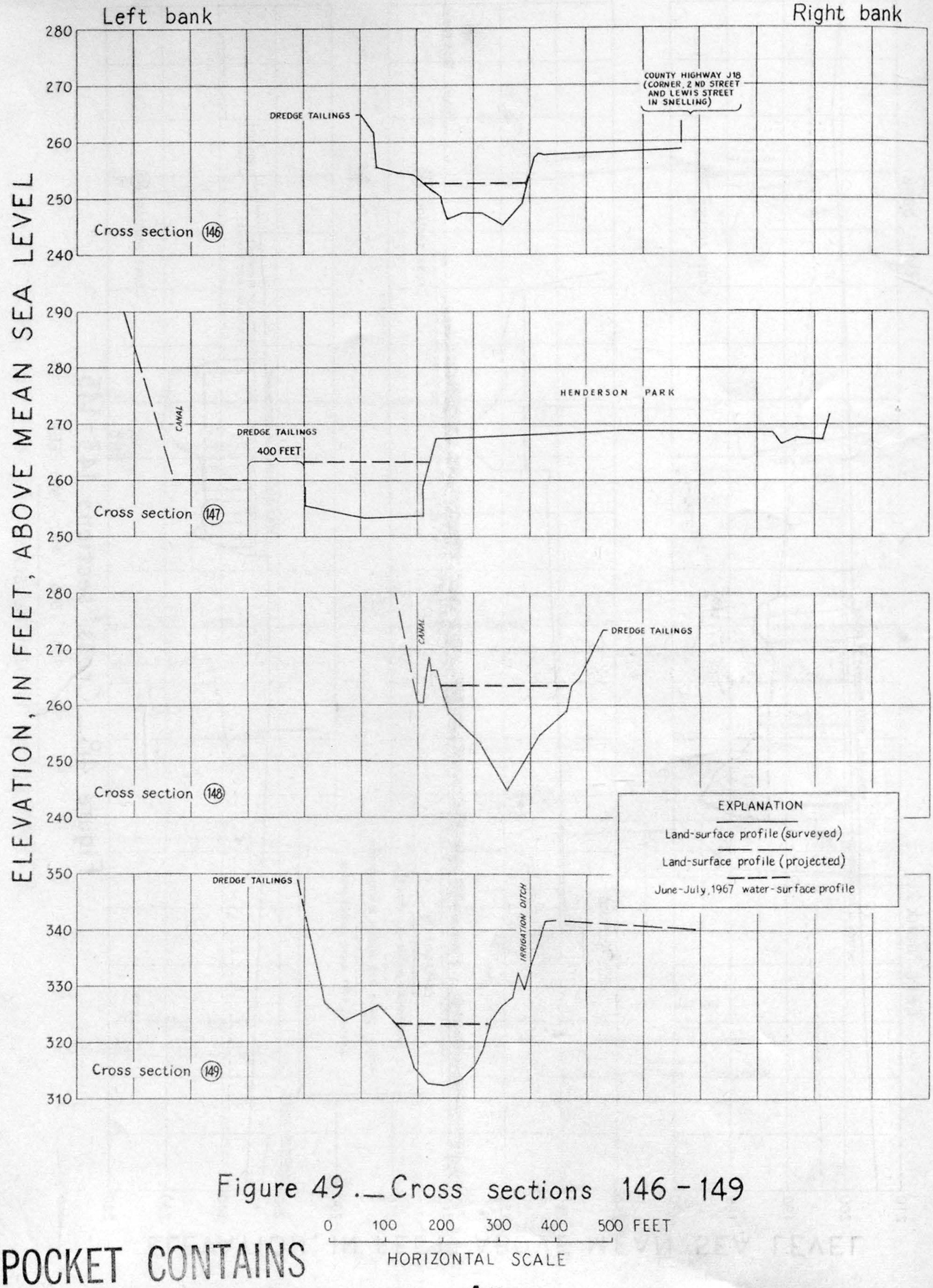


Figure 49.—Cross sections 146-149

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