CAPACITY OF THE DIVERSION CHANNEL BELOW THE FLOOD-CONTROL DAM ON THE BIG LOST RIVER AT THE IDAHO NATIONAL ENGINEERING LABORATORY, IDAHO

by C. M. Bennett

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CONVERSION FACTORS

For readers who prefer to use metric (International System) units, conversion factors for inch-pound units used in this report are listed below.

Multiply inch-pound units	By	To obtain metric unit
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer
acre-foot (acre-ft)	1,233	cubic meter
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

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ABSTRACT

Stage-discharge relations were computed for two selected cross sections of a diversion channel at the Idaho National Engineering Laboratory for discharges between 2,000 and 7,200 cubic feet per second. The channel diverts water from the Big Lost River into four spreading areas where the water infiltrates into the ground or evaporates. Computed water-surface profiles, based on channel conditions in the summer of 1985, indicate that the diversion channel will carry a maximum discharge of 7,200 cubic feet per second from the Big Lost River into the first spreading area. Backwater from the spreading areas is not expected to decrease the carrying capacity of the diversion channel. An additional 2,100 cubic feet per second will pass through two low swales west of the main channel for a combined maximum diversion capacity of 9,300 cubic feet per second.

INTRODUCTION

The diversion channel from the Big Lost River at the Idaho National Engineering Laboratory (INEL) is used to regulate the flow of the Big Lost River. Regulation is needed in order to minimize the probability of inundating several nuclear-reactor facilities, a radioactive waste-disposal and storage area, and the many support facilities that are located on the floodplain of the Big Lost River (fig. 1). The capacity of the diversion channel is needed to evaluate the potential for flooding from snowmelt or a failure of Mackay Dam, which is about 30 mi northeast of Arco.

The need for flood control at the INEL has been recognized since the early 1950's when the Test Reactor Area and the Idaho Chemical Processing Plant were threatened by localized flooding that occurred because of ice jams in the Big Lost River. A small diversion dam was constructed across the Big Lost River in 1958 to divert water from the river through the diversion channel into a spreading area. Repeated threats of flooding in the late 1960's, early 1970's, and early 1980's occurred when the Big Lost River filled Playas 1 and 2 and overflowed into Playa 3 near the Loss-of-Fluid Test facility (fig. 1). High streamflow and air temperatures as low as -47° F in the winter of 1983-84 caused ice jams that imposed a danger of localized flooding. The diversion channel was enlarged in 1984 to provide additional flood control; the dam across the Big Lost River, and the containment dikes along the diversion channel and spreading areas were raised several feet. The study was conducted by the U.S. Geological Survey in cooperation with the U.S. Department of Energy. The results of this study will be used in a larger study conducted by EG&G Idaho, Inc.--a contractor to the Department of Energy, Idaho Operations Office.

Purpose and Scope

The purpose of this study was to determine: (1) the capacity of the diversion channel subsequent to raising the elevation of the containment dikes; and (2) the theoretical stage-discharge relation for discharges greater than 2,000 ft^3/s at a gaging station in the channel between the dam across the Big Lost River and the spreading areas. This report describes:

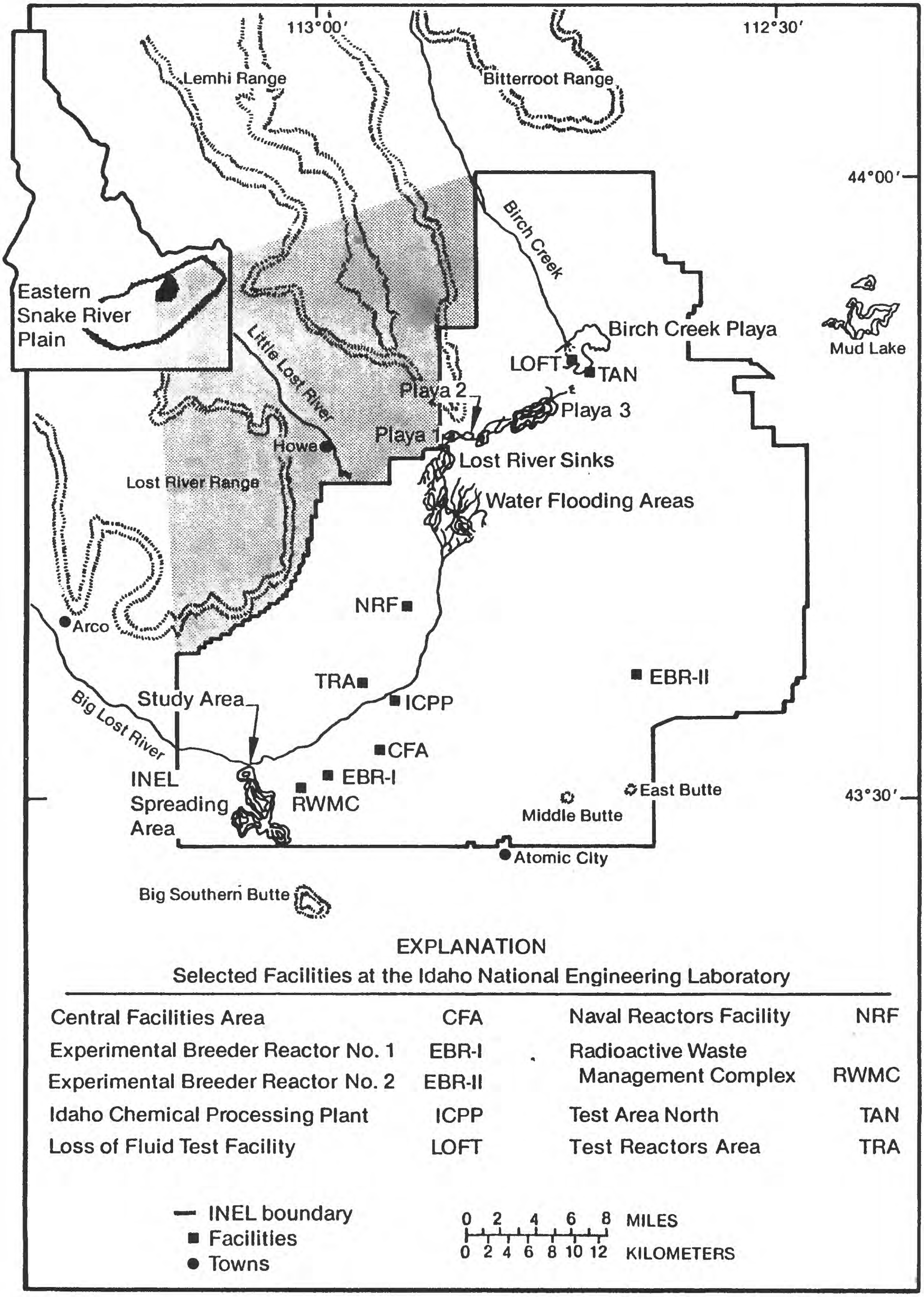

Figure 1.--Selected facilities and surface-water features in the vicinity of the Idaho National Engineering Laboratory.

(1) the channel geometry and roughness coefficients for 11 cross sections in the upstream most 1,500 ft of the diversion channel; (2) the computation of water-surface elevations and capacity of the diversion channel; (3) the capacity of two bypass swales that would intercept and route water into the spreading areas before the containment dikes would be topped; and (4) the stage-discharge relation at the gaging station and a cross section at the diversion dam.

Location

The diversion channel is in the southwestern part of the INEL, about 4 mi east of the boundary (fig. 1). Streamflow in the Big Lost River originates in the high mountain valleys and peaks at elevations that range from about 5,500 ft near Arco to over 12,000 ft in the Lost River Range. After leaving the mountain valleys and entering the Snake River Plain, the Big Lost River departs from a southeasterly direction and flows in a large arc toward the north. The diversion is at the southernmost point of this large arc. Drainage area of the the Big Lost River upstream from the diversion dam is about 1,450 mi^2.

DESCRIPTION OF THE DIVERSION CHANNEL

The diversion channel was excavated through several basalt ridges and intervening surficial sedimentary deposits to connect the Big Lost River with a series of natural depressions. The depressions are designated as spreading areas A, B, C, and D (fig. 2). Water is diverted into the diversion channel by a low earthen dam across the Big Lost River (fig. 3). The dam is part of a long, continuous dike along the left side of the river and diversion channel. Two 6-ft-diameter corrugated metal pipes permit passage of less than 900 ft^3/s through the dam into the river (Lamke, 1969, p. 14). Flow in the river is regulated by gates on the culverts. During floods, flow in excess of that allowed to pass through the culverts is carried by the diversion channel. Flow in the diversion channel is uncontrolled at discharges that exceed the capacity of the culverts.

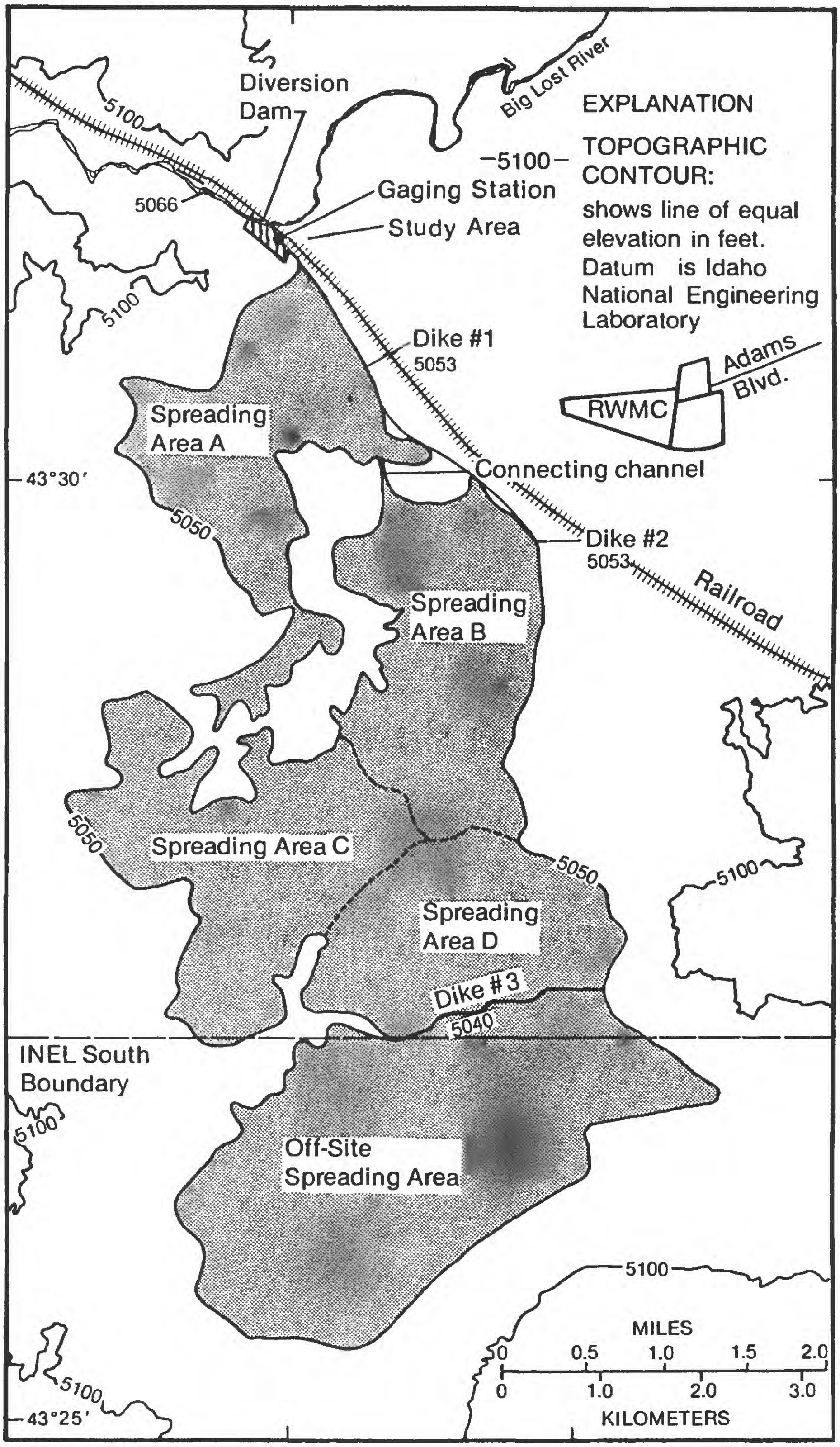

Figure 2.--Location of the study area and enlarged spreading areas A, B, C, and D

Figure 3.--Downstream view of diversion channel at diversion dam.

The diversion channel extends about 0.9 mi from the point of diversion to spreading area A (fig. 2). Water flows from spreading area A through a short connecting channel into the three other spreading areas. McKinney (1985) reports the capacity of the spreading areas to exceed 58,000 acre-ft. He also reports the elevation of the top of the dike that contains spreading areas A and B on the east to be 5,053 ft, INEL datum. All elevations in this report are to sea level datum unless otherwise specified; to convert elevations to INEL datum add 1.29 ft to sea level datum.

The configuration of the channel is unusually irregular and rough. Resistant basalt ridges create an irregular channel bottom which cause riffles and waterfalls at low to medium stages. The containment dike forms the left bank of the diversion channel. Basalt boulders, up to 5 ft in diameter, serve as rip-rap along the lower part of the dike. The upper part of the dike is predominately gravel. The dike is largely devoid of brush and other vegetation. Scalloped areas, depressions, and basalt ridges form the right bank. The right-bank overflow section is sparsely to moderately covered with vegetation, chiefly sagebrush and grass.

ANALYTICAL PROCEDURES

The Federal Highway Administration-U.S. Geological Survey Bridge Waterways Analysis Model, WSPRO, (Shearman and others, 1986) was used to compute water-surface profiles. Estimates of the theoretical capacity for the diversion channel and bypass swales were obtained from these profiles. The model applies standard step-backwater techniques to open-channel flow. Data necessary for the computations include the geometry of the channel, channel-roughness coefficients, and an initial water-surface elevation. These data are used to compute water-surface profiles that correspond to known or assigned discharges. A detailed discussion of the hydraulic principles and assumptions used in the evaluation of step-backwater analyses is presented by Chow (1959) and Davidian (1984). Specifics of the WSPRO program are described by Shearman and others (1986).

COLLECTION OF FIELD DATA

Field Survey

The geometry of the stream channel, overflow sections and bypass swales was determined by transit-stadia survey. The survey was run using the basic techniques as described by Benson and Dalrymple (1967) for indirect-discharge measurements. Channel cross sections were surveyed in August 1985 by R. L. Backsen and C. M. Bennett at selected intervals throughout the study reach. A plan view of the positions and length of the cross sections and overflow swales is shown in figure 4. Section 3 was surveyed at an angle to the flow because the main channel makes a sharp angle bend between two large rocks. An adjustment for angularity was made in the computations. The configurations of the channel cross sections and overflow swales are shown in figures 5 to 17. Some sections exceed the width that can be plotted at the selected scale and therefore appear to be truncated before the section reaches an elevation equal to that of the maximum water-surface elevation.

Roughness coefficients

The channel roughness coefficient used in backwater computations, is affected primarily by:

(1) Bed roughness
(2) Cross-section irregularities
(3) Depth of flow
(4) Vegetation, and
(5) Channel alignment

Roughness coefficients were assigned on the basis of field observations by B. N. Aldridge, C. M. Bennett, and L. J. Mann. The irregular geometry and rough nature of the channel are outside the range for which variables of roughness coefficients have been defined by research on other streams (written communication, B. N. Aldridge, Sept. 1985). The coefficients assigned to the individual cross sections and subsections are given in Table 1. The diversion channel was divided into two subareas--the main

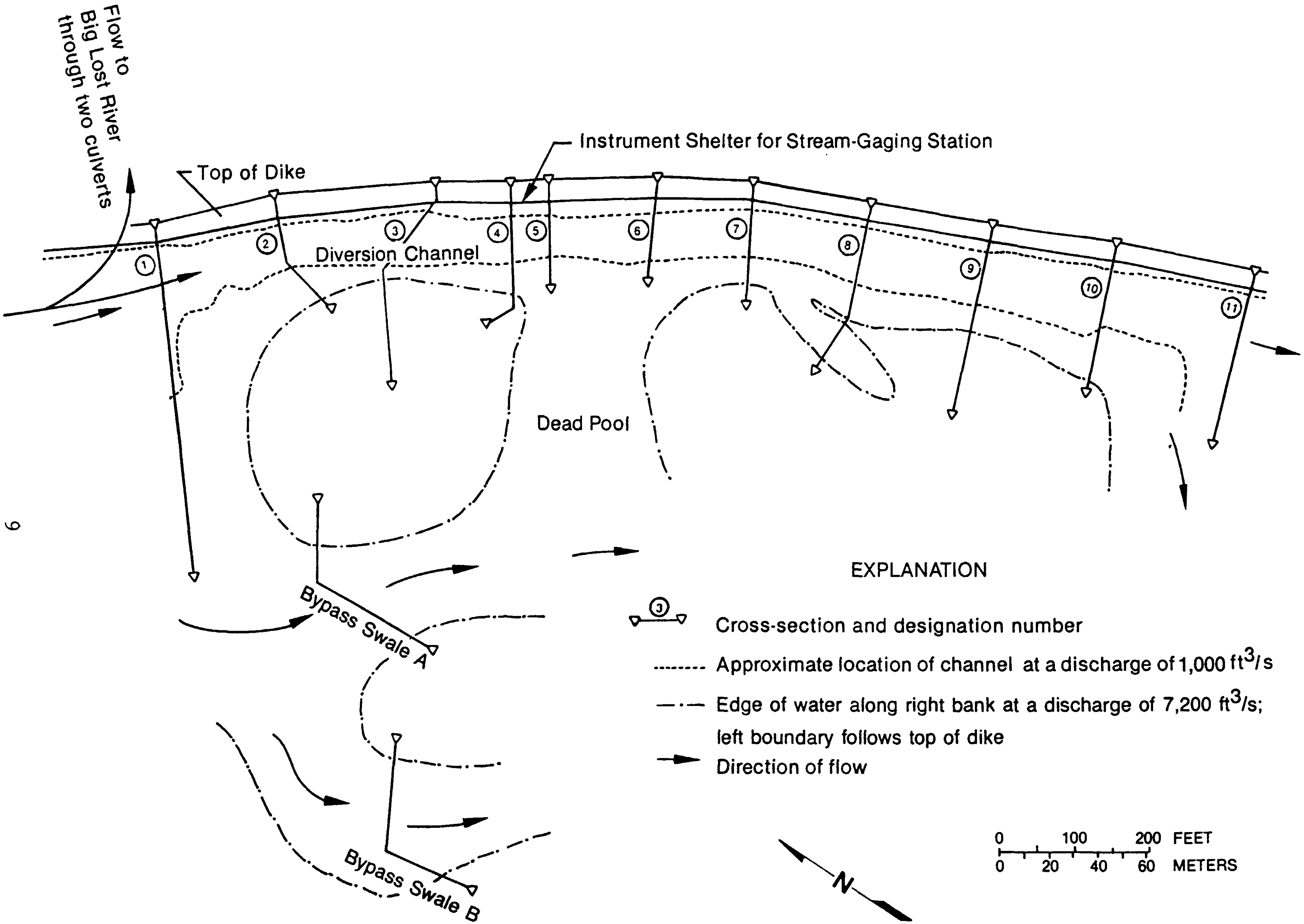

Figure 4.--Plan view of diversion channel showing locations of cross sections 1-11 and bypass swales A and B.

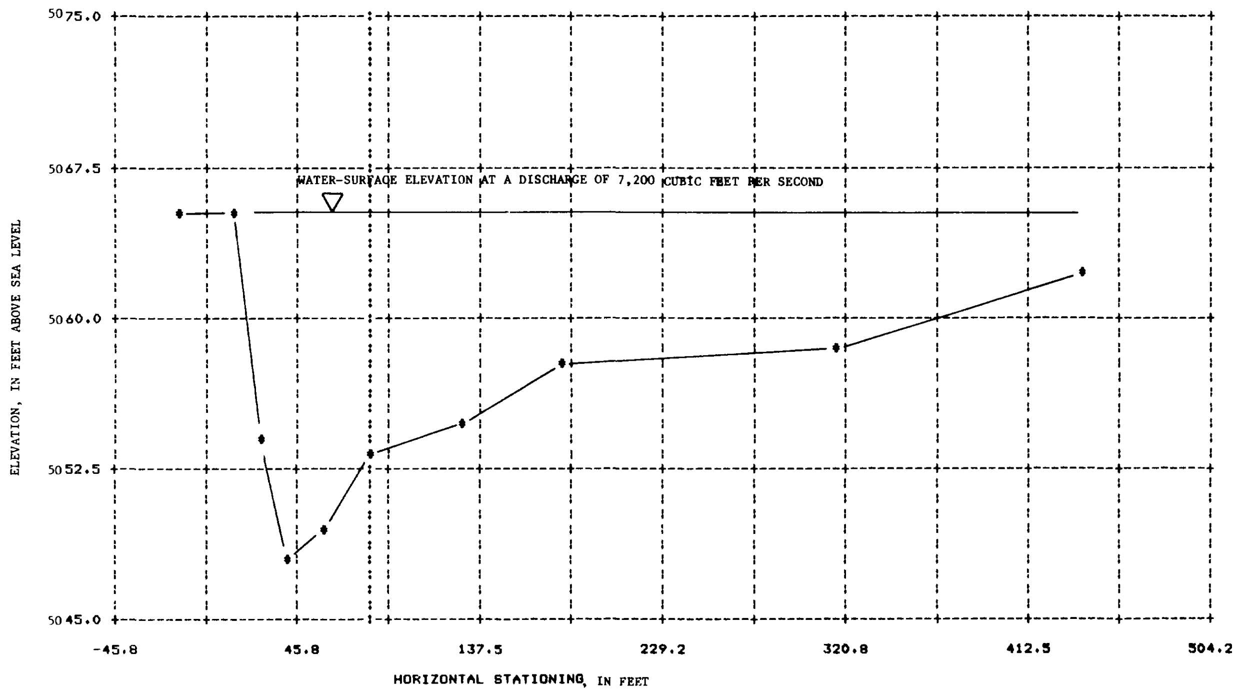

Figure 5.--Configuration of the diversion channel at cross section 1.

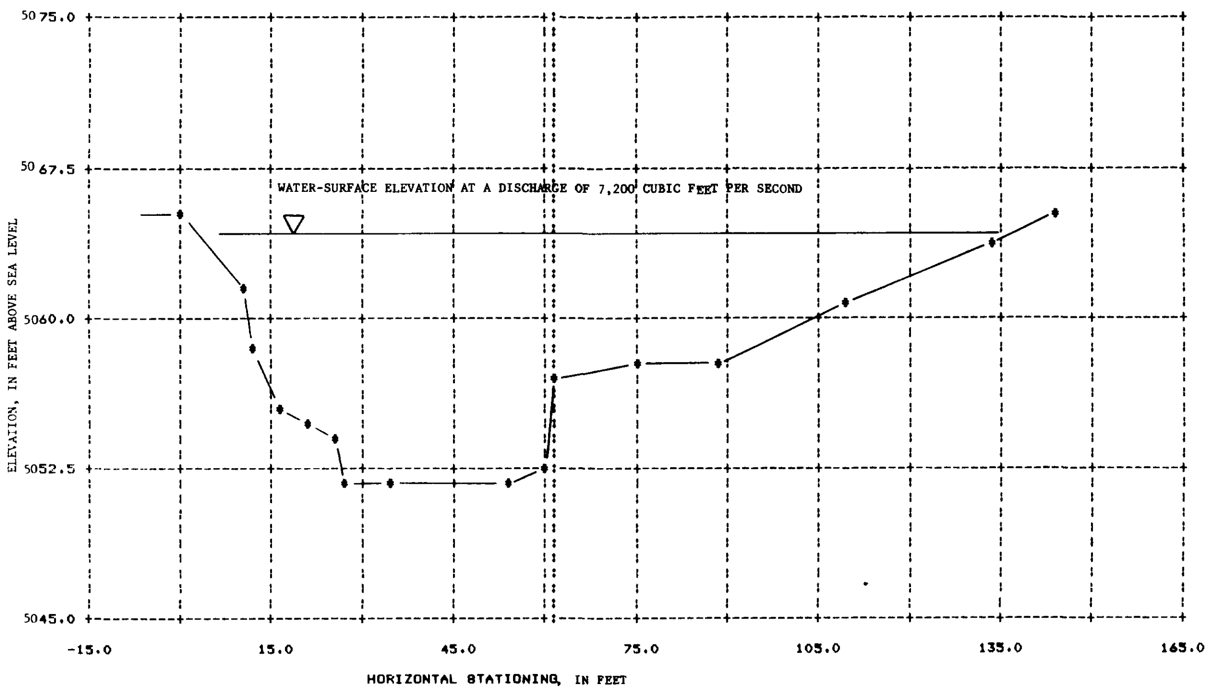

Figure 6.--Configuration of the diversion channel at cross section 2.

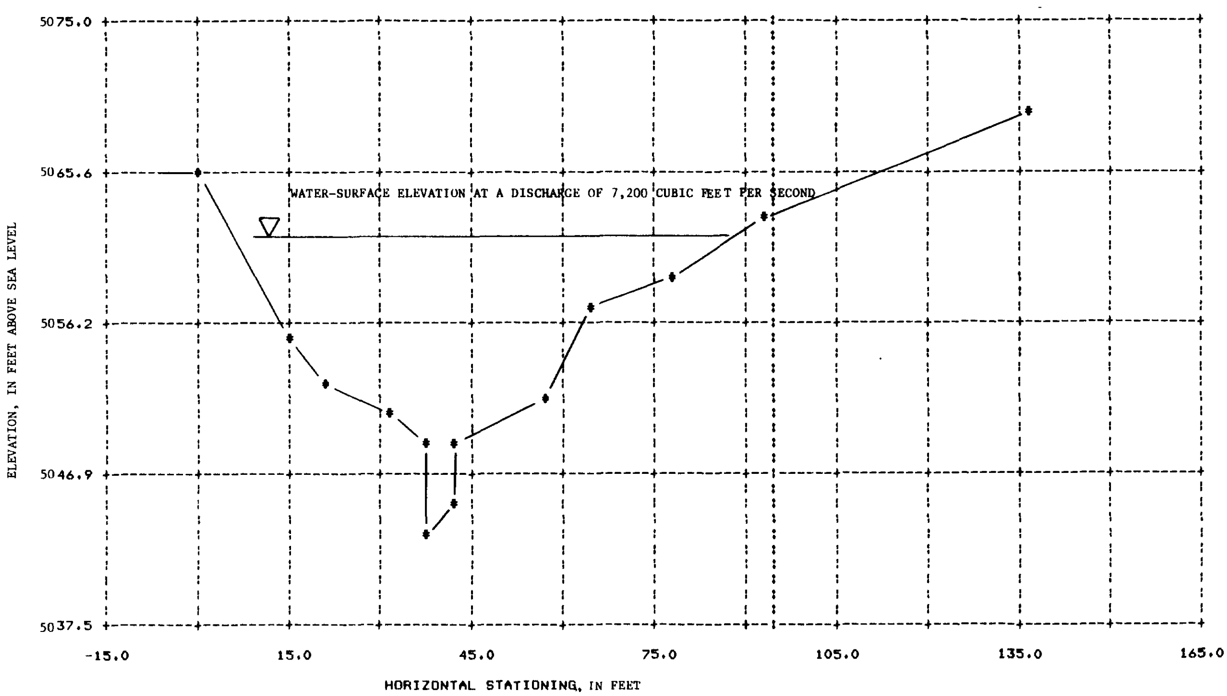

Figure 7.--Configuration of the diversion channel at cross section 3.

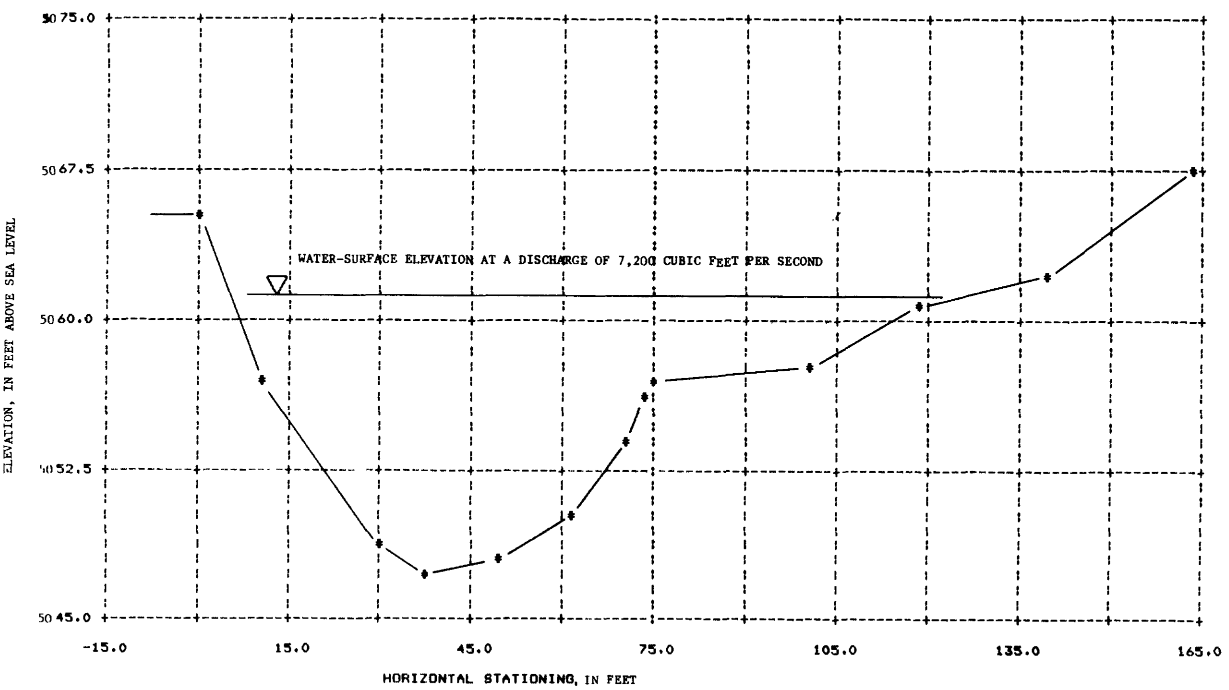

Figure 8.--Configuration of the diversion channel at cross section 4.

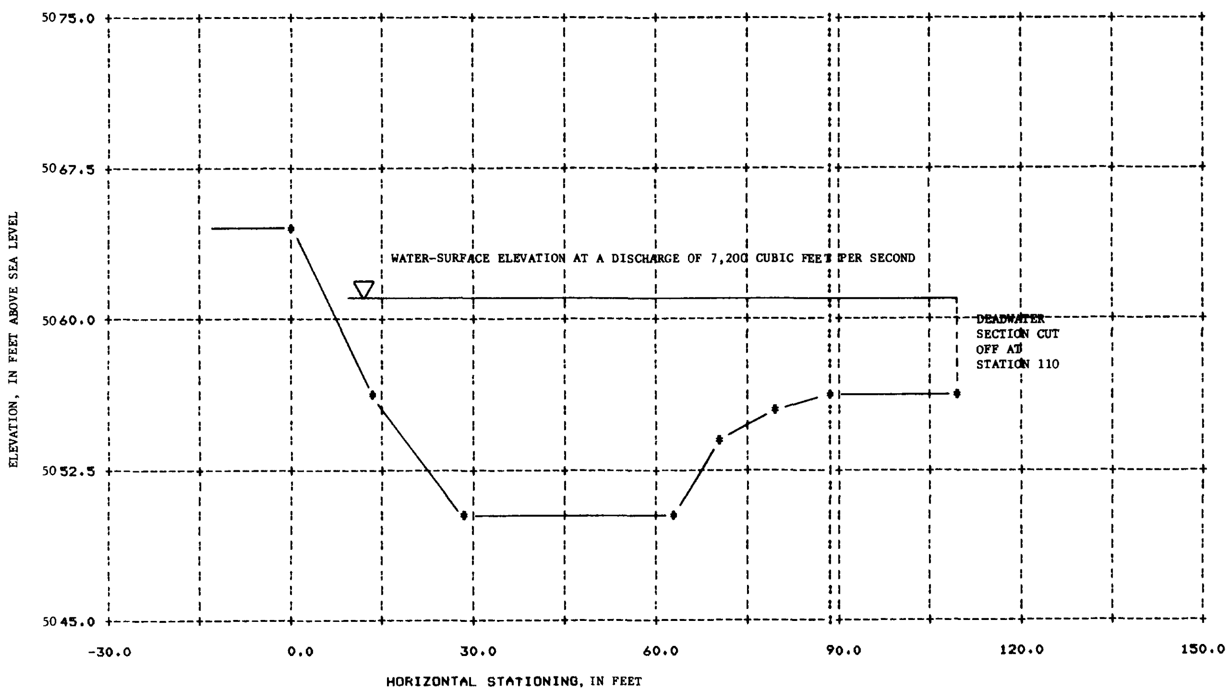

Figure 9.--Configuration of the diversion channel at cross section 5.

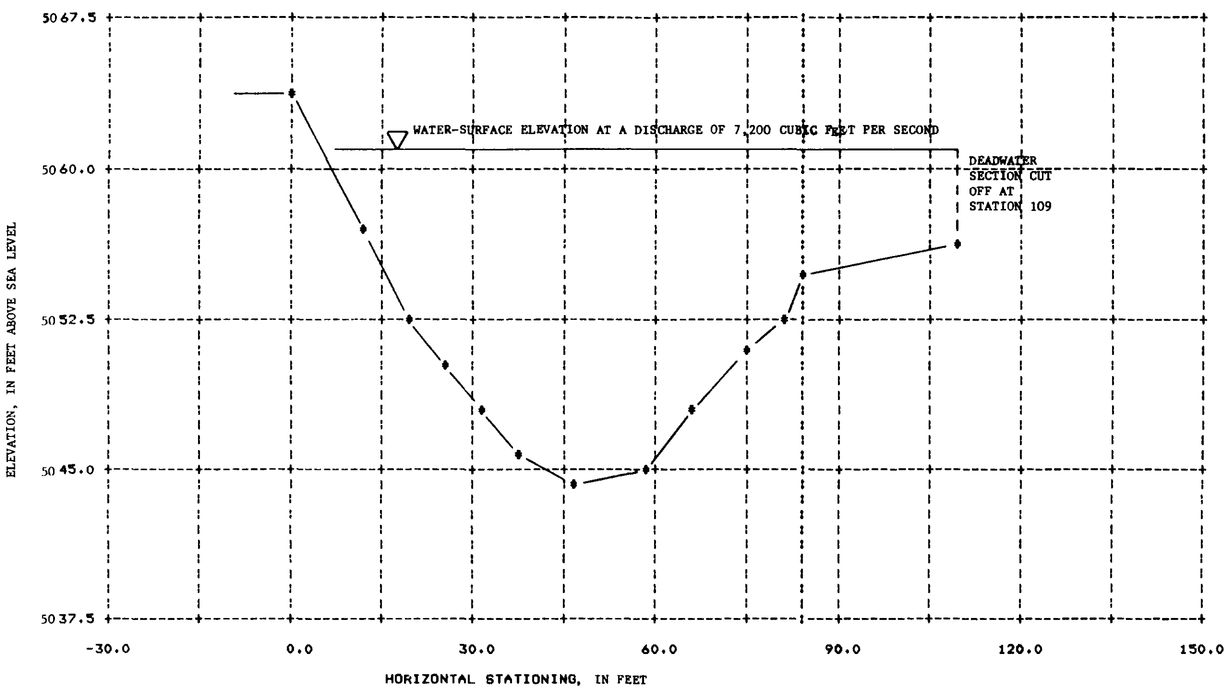

Figure 10.--Configuration of the diversion channel at cross section 6.

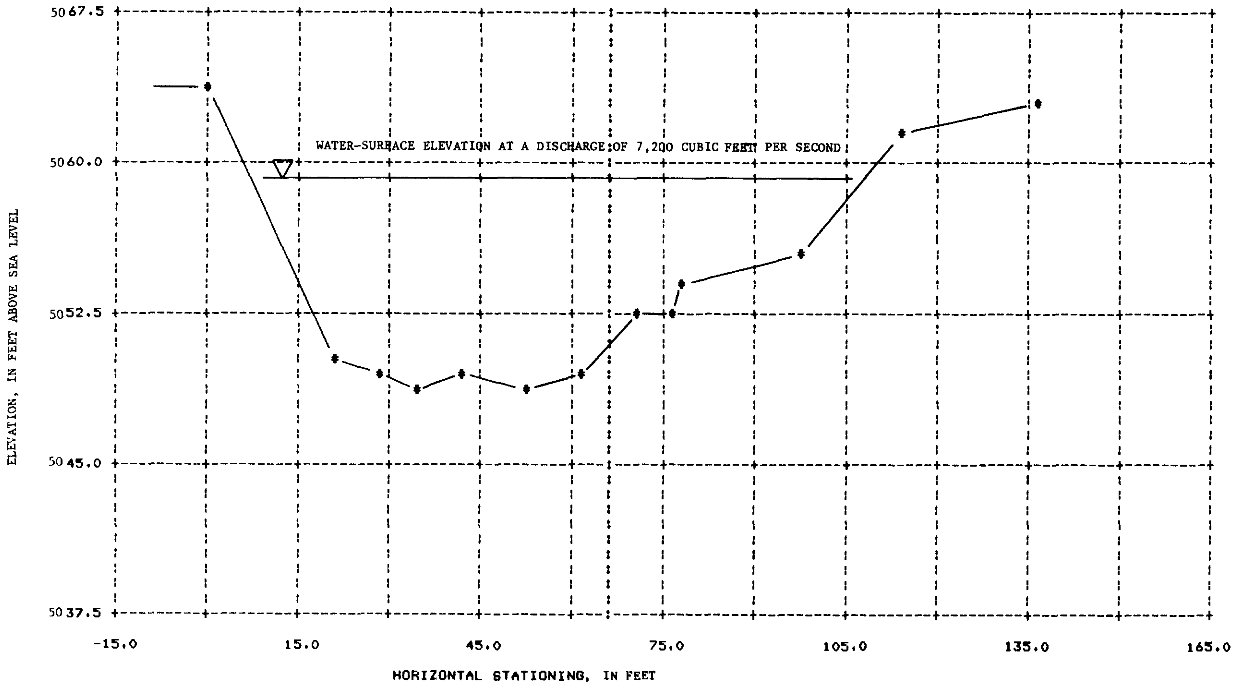

Figure 11.--Configuration of the diversion channel at cross section 7.

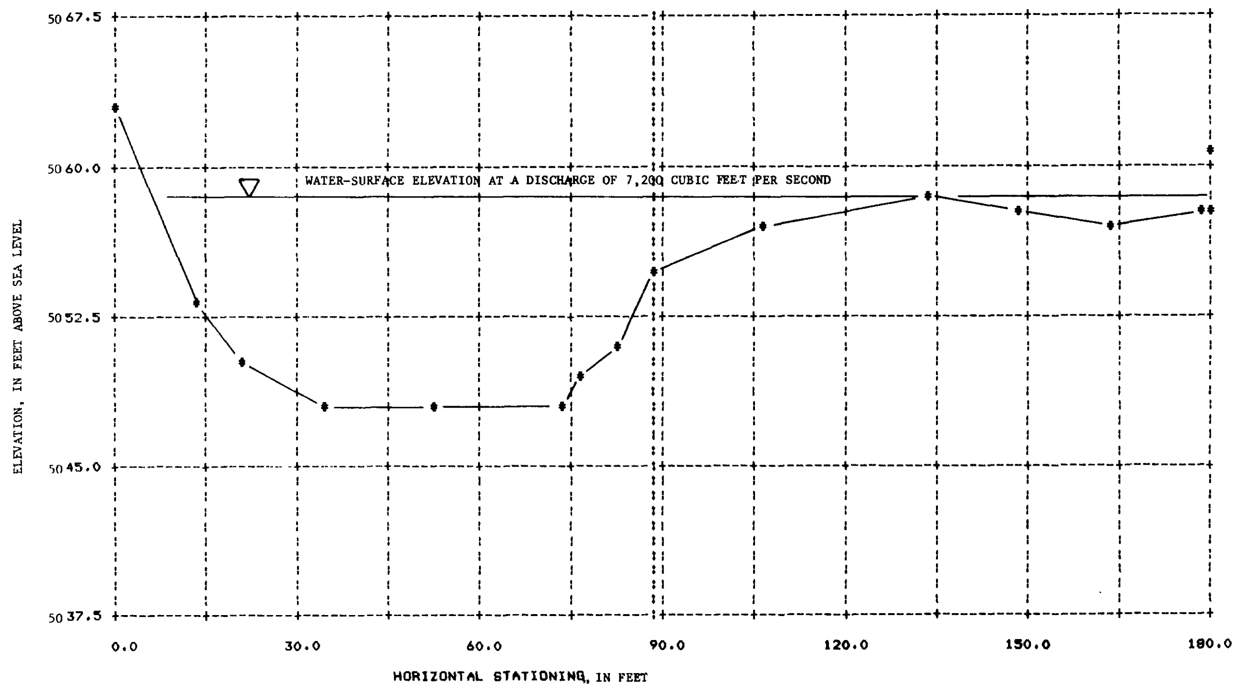

Figure 12.--Configuration of the diversion channel at cross section 8.

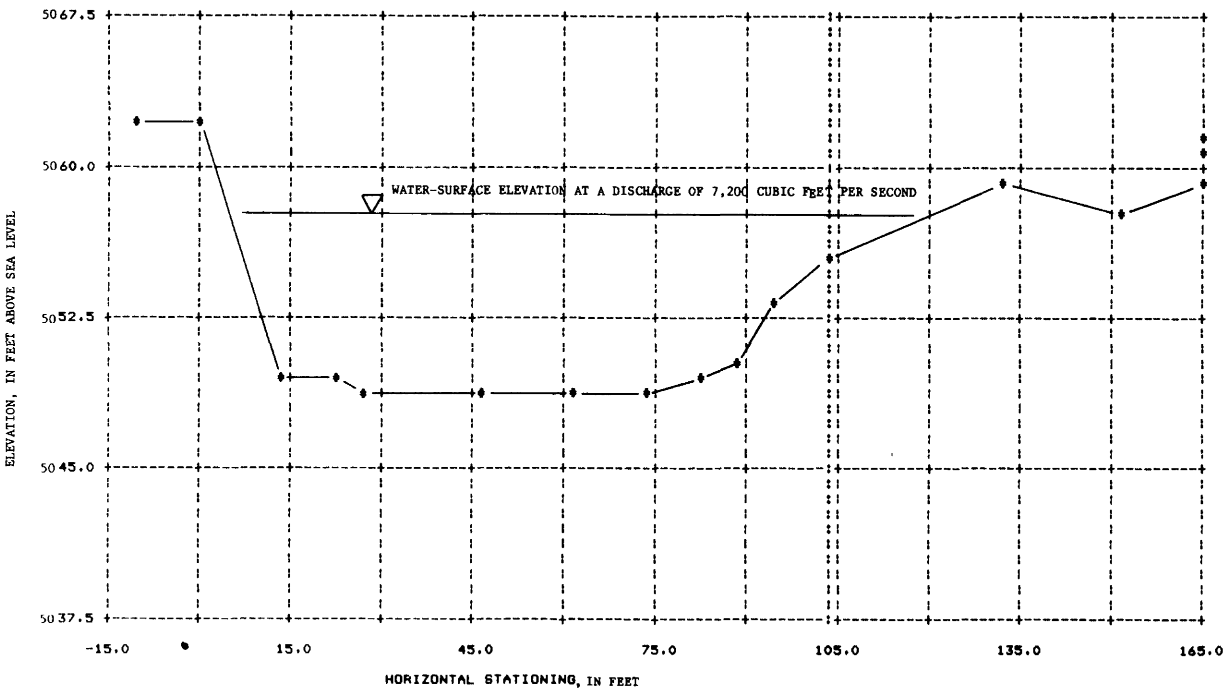

Figure 13.--Configuration of the diversion channel at cross section 9.

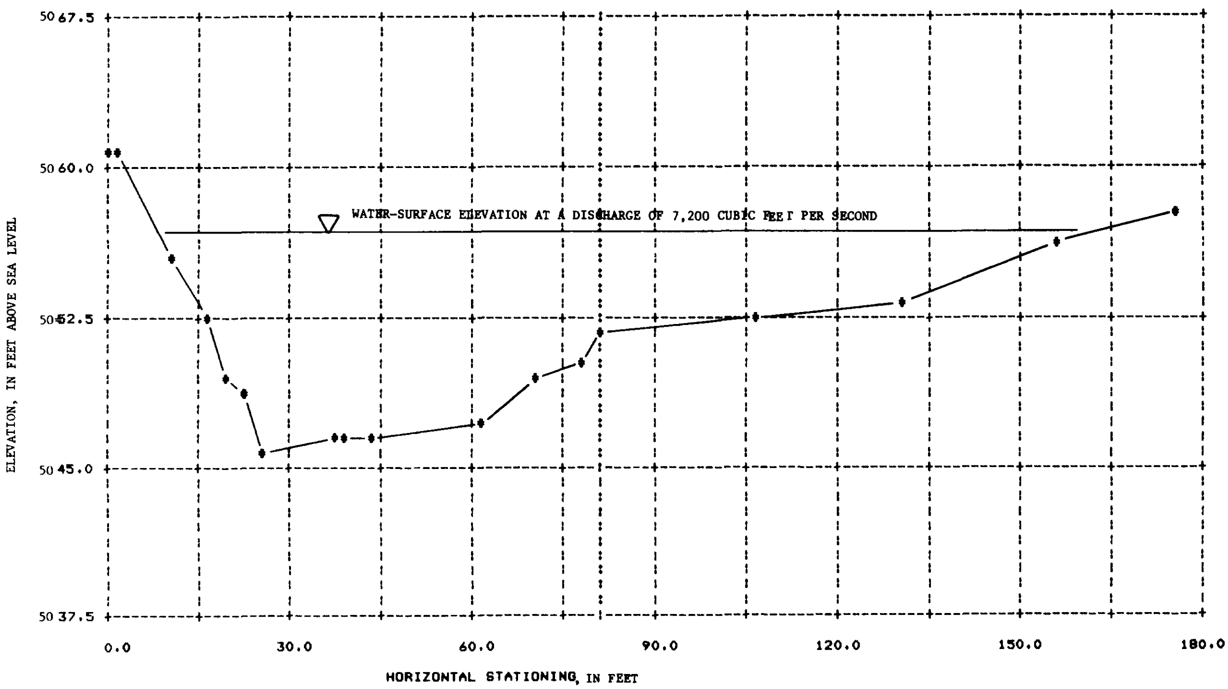

Figure 14.--Configuration of the diversion channel at cross section 10.

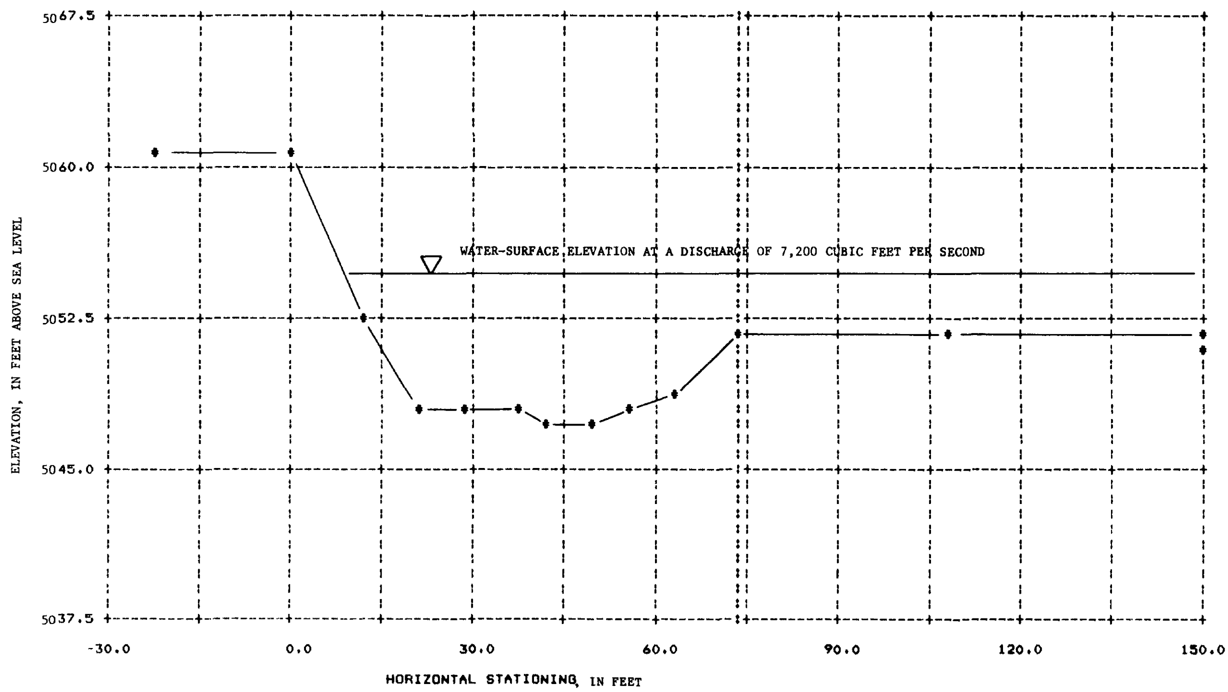

Figure 15.--Configuration of the diversion channel at cross section 11.

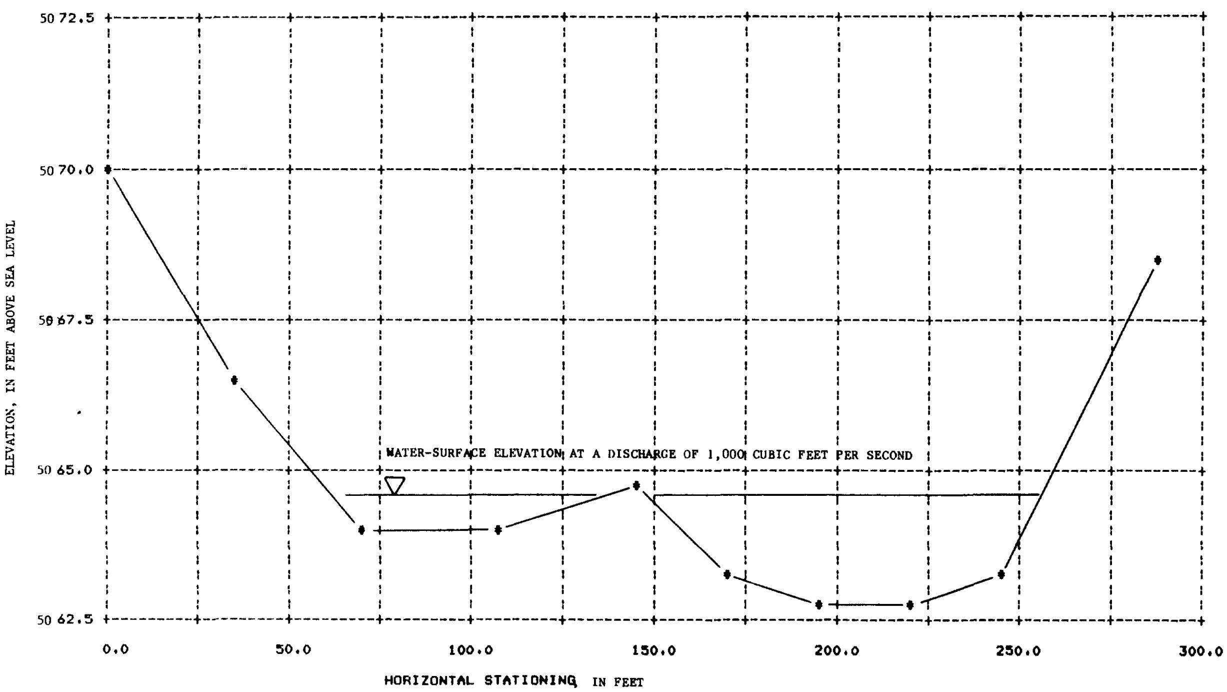

Figure 16.--Configuration of channel at bypass swale A.

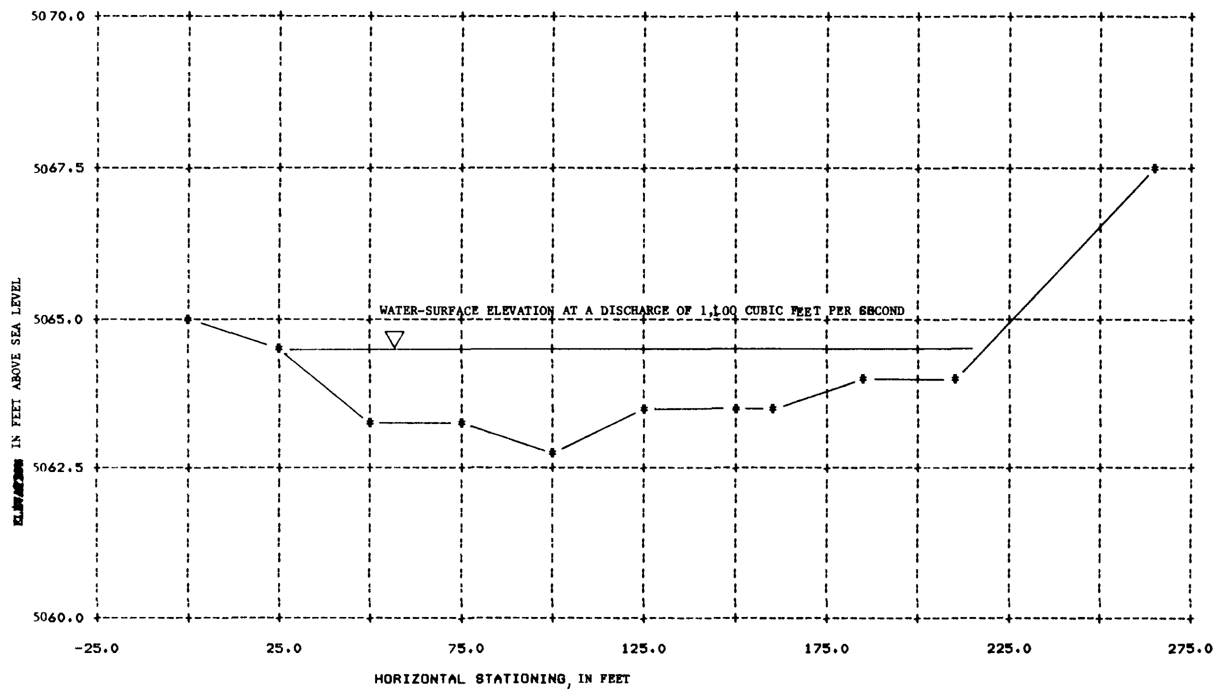

Figure 17.--Configuration of channel at bypass swale B.

Table 1.--Assigned roughness coefficients for diversion channel

Cross section	Main channel		Right-overflow subarea	
	Hydraulic depth (feet)	Roughness coefficient	Hydraulic depth (feet)	Roughness coefficient
1	< 4	0.040	< 2	0.080
	> 7	.035	> 5	.055
2	< 2	.070	< 2	.060
	> 5	.060	> 4	.050
3	< 2	.050	< 2	.065
	> 5	.050	> 4	.055
4	< 2	.040	< 2	.055
	> 4	.040	> 7	.055
5	< 2	.040	< 3	.070
	> 4	.040	> 6	.055
6	< 4	.060	< 2	.080
	> 7	.048	> 6	.055
7	< 2	.060	< 2	.080
	> 4	.045	> 6	.055
8	< 2	.060	< 2	.080
	> 5	.045	> 5	.055
9	< 2	.060	< 2	.080
	> 4	.045	> 5	.055
10	< 2	.045	< 2	.070
	> 4	.045	> 6	.050
11	< 2	.060	< 2	.070
	> 4	.050	> 5	.055

channel and the right-bank overflow. Two roughness coefficients are given for each subarea. The first is used when the hydraulic depth (area of subarea divided by top width of subarea) is less than that specified. The second is used when the hydraulic depth is greater than that specified. For example, the roughness coefficient for the main channel in cross section 1 is 0.040 if the hydraulic depth is less than 4 ft and 0.035 if is greater than 7 ft. Roughness coefficients for hydraulic depths between upper and lower assigned depths are determined by straight-line interpolation.

COMPUTATION OF WATER-SURFACE PROFILES

The WSPRO Computer Program (Shearman, 1986) was used to compute water-surface profiles in the diversion channel for selected discharges between 2,000 ft^3/s and 7,200 ft^3/s. Three step-backwater computations were made to identify the control section and to test the sensitivity of the computed water-surface elevation at section 1 to changes in starting locations and initial water-surface elevations; initial computations indicated that the containment dike would first be overtopped by high flows in the diversion channel at section 1. Two of the step-backwater computations were started from section 11 at different initial water-surface elevations, and one was started from section 5, which is at a concrete broad-crested weir that is the control for a gaging station located at section 4.

Sections 5 and 11 were selected as the initial starting section because of the irregularities in the channel bottom below these sections. Sharp drops occur in the elevation of the streambed below sections 2, 5, and 11. Between sections 2 and 3 the streambed elevation drops about 9 ft and between sections 5 and 6 it drops about 6 ft (fig. 18). Immediately downstream from section 11, the elevation of the streambed drops in excess of 5 ft at a waterfall. From the waterfall, the streambed drops over several rock ledges before reaching spreading area A. Step-backwater computations were made assuming that critical depth occurred at the initial starting section. The initial water-surface elevation selected for each step-backwater computation is the elevation which produces minimum specific energy at the initial section.

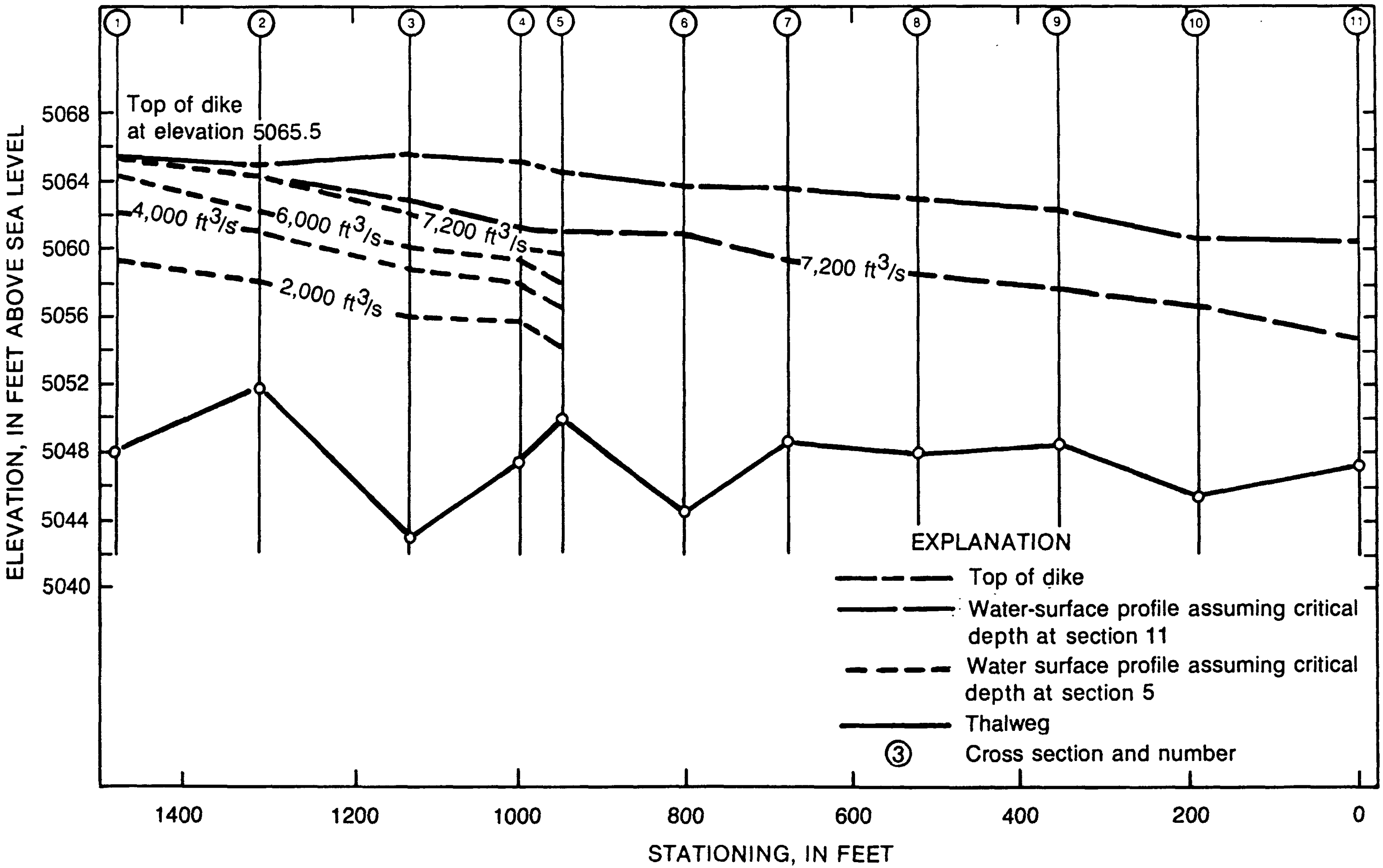

Figure 18.--Profile of top of dike, computed water-surface elevations, and streambed elevations upstream from section 11.

The right bank overflow section expands markedly between sections 10 and 11. At section 11 flow could extend over several hundred feet of width. In the field, it appeared that water would not have free access to much of the overflow section. The cross section was terminated 135 ft from the right edge of the main part of the diversion channel; the end of the section was directly downstream from high ground at section 10. At section 11, two initial water-surface elevations were used with each discharge because of an uncertainty of the right bank overflow section to carry water. The lower of the two initial water-surface elevations produced minimum specific energy for the main channel plus the 135-ft wide overflow section. The higher initial water-surface elevation produced minimum specific energy if all flow were confined to the main channel. The higher initial water-surface elevation represents the highest probable water-surface elevation that would occur at section 11. For each selected discharge, the two computed water-surface elevations differ by about 1 ft at section 11, but the computed water-surface elevations at section 1 differ by about 0.1 ft.

Each of the two step-backwater computations beginning at section 11 show subcritical flow at all upstream sections for all discharges. However, the computed water-surface elevation for section 4 at 1,500 ft^3/s was at least 0.8 ft higher than the elevation recorded at the gaging station for a measured discharge of 1,530 ft^3/s. The above computations indicated flow at section 5 was subcritical, but because of the disparity between computed and recorded elevations at section 4 a third step-backwater computation was made assuming critical depth at section 5. The water-surface elevation computed for section 4 in the third computation agrees with recorded elevation, indicating that critical flow does occur at section 5. Therefore, sections 6 through 11 do not need to be considered in the analysis. This computation indicated subcritical flow at sections 1 through 4.

The water-surface elevation at section 1 calculated in the third step-backwater computation was within a few hundredths of a foot of those calculated in the first two computations. A comparison of the water-surface profiles for the three computations indicate that the calculated water-surface elevation at section 1 is insensitive to starting location and initial water-surface elevations at sections 5 and 11. Water-surface elevations from the computations starting from section 5 are given in table 2 and are used in subsequent analyses in this report.

Table 2.-- Computed water-surface elevations at cross sections 1-5 for selected discharges.
(Profiles start from critical elevations at Section 5.)

Section	Elevation of top of dike in feet	Elevation of streambed in feet	Computed water-surface elevation in feet for the indicated discharge in cubic feet per second. 2000	4000	6000	7000	7200
01	5065.5	5048.0	5059.3	5062.2	5064.4	5065.2	5065.5
02	5065.0	5051.8	5058.1	5061.0	5063.2	5064.0	5064.3
03	5065.6	5043.0	5056.2	5058.7	5060.2	5060.9	5061.1
04	5065.2	5047.4	5055.7	5057.9	5059.3	5059.9	5060.1
05	5064.5	5049.9	*5054.2	*5056.6	*5058.0	*5058.6	*5058.7

*Elevation at which critical flow occurs; used as starting elevation for step-backwater computation.

A check was made to determine if a high water-surface elevation in spreading area A could cause backwater upstream from section 5. The potential for backwater is extremely small. The streambed immediately downstream from section 11 is about 8 ft lower than the crest of the control at section 5, and downstream from section 11, there are several more drops in the streambed. Water will begin to flow out of spreading area A at an elevation that is 14 ft lower than the crest of the control at section 5. The maximum elevation to which water can rise in spreading area A, under present conditions, is about 5,051.7 ft above sea level--the elevation at the top of the containment dike at the spreading area. The initial water-surface elevation selected in the step-backwater program for all discharges greater than 2,000 ft^3/s is above that elevation. Thus, it is unlikely that backwater from spreading area A could submerge either section 11 or section

5 or that it could change the shape of the computed water-surface profiles upstream from section 5.

STAGE-DISCHARGE RELATION

Water-surface elevations computed by the step-backwater program were used to define the relation between stage--the water-surface elevation--and discharge at the cross sections 1 and 4 (fig. 19). The relation of stage to discharge at cross section 1 is crucial because this is the point where flow will first occur across the containment dike. Cross section 4 is at the gaging station. The relation developed at section 4 using the step-backwater program closely approximates a logarithmic extension of the stage-discharge relation developed for the gaging station using current meter measurements.

Lamke (1969, p. 15), through use of a different step-backwater program from that used in this study, defined a stage-discharge relation at the intakes of the former stream-gaging station. The intakes were located in the concrete weir at section 5. Lamke's study indicated subcritical flow at section 5, but the current-meter measurements that now define the stage-discharge relation for the gage were not available to check his computations. After adjusting for datum corrections, water-surface elevations computed at section 5 by Lamke for discharges between 2,000 and 3,500 ft^3/s are nearly identical to the water-surface elevations from the step-backwater computation that started at Section 11 in this study. Lamke (1969) extended his curve to 3,500 ft^3/s because that was the capacity of the channel at the time.

DISCHARGE OF THE BYPASS SWALES

When the discharge approaches 6,000 ft^3/s, water will begin to flow through two topographically low swales that bypass the diversion channel. The two swales are about 400 and 800 ft, respectively, west of the main channel and are designated as bypass swales A and B on figure 4. Surveyed

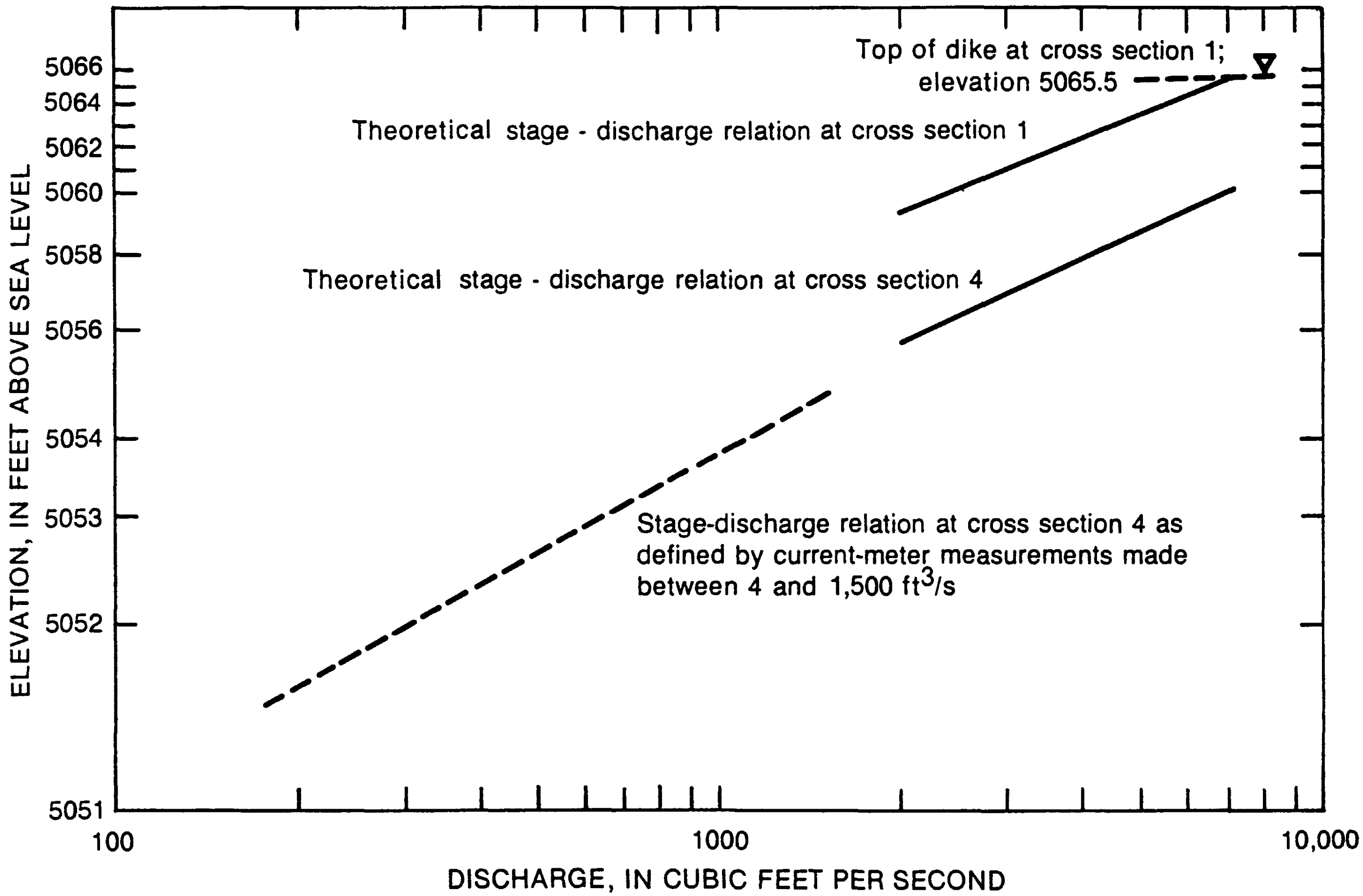

Figure 19.--Stage-discharge relation for cross sections 1 and 4.

cross-sectional area of the swales and data from field observations were used in a step-backwater program to compute profiles for several discharges. The discharges in bypass swales A and B were calculated to be 1,000 ft^3/s and 1,100 ft^3/s, respectively when the stage at section 1 is 5,065.5 ft. An independent check using a flow-over-an-embankment type computation as described by Hulsing (1967, p. 26), showed computations using the step-backwater method to be reasonable.

CAPACITY OF THE DIVERSION CHANNEL

Water will flow over the dike at section 1 before water reaches the top of the dike along the rest of the diversion channel. The computed water-surface profiles (fig. 18) show that a discharge of 7,200 ft^3/s will occur when the stage at cross section 1 is at an elevation of 5,065.5 ft--the same elevation as that of the top of the dike. It is estimated that at this stage an additional 2,100 ft^3/s will bypass the study reach through the two swales. Total capacity of the diversion channel and bypass swales is 9,300 ft^3/s. The accuracy limits of the computational procedures are probably on the order of plus or minus about 10-15 percent. A sustained flow at or above this discharge could damage or destroy the dike. Lowering the stage at section 1 by a few tenths of a foot would greatly reduce the amount of water in the bypass swales.

SUMMARY

This study evaluates the capability of a diversion channel at the Idaho National Engineering Laboratory to carry flood water from the Big Lost River into spreading area A. A theoretical stage-discharge relation was developed for the gaging station on the diversion channel for discharges between 2,000 and 7,200 ft^3/s.

Computed water-surface profiles, based on channel conditions in the summer of 1985, indicate the combined capacity of the diversion channel and two bypass swales is 9,300 ft^3/s. At that discharge, 7,200 ft^3/s will flow

through the diversion channel and the elevation of the water surface will be at the top of the dike at the upstream end. At the same time, 2,100 ft^3/s will bypass the diversion channel through two topographically low swales west of the channel. A sustained flow at or above 9,300 ft^3/s could damage or destroy the dike banks by erosion. Overflow will first top the containment dike at cross section 1, located near the downstream control structure on the diversion dam. Backwater that results from high stages in spreading area A is not expected to affect the carrying capacity of the diversion channel.

REFERENCES CITED

Benson, M. A. and Dalrymple, Tate, 1967, General field and office procedures for indirect discharge measurements: U.S. Geological Survey Techniques of Water-Resources Investigations, Book 3, Chapter A1, 30 p.

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

Davidian, Jacob, 1984, Computation of water-surface profiles in open channels: U.S. Geological Survey Techniques of Water-Resources Investigations, Book 3, Chapter A15, 48 p.

Hulsing, Harry, 1967, Measurement of peak discharge at dams by indirect methods: U.S. Geological Survey Techniques of Water-Resources Investigations, Book 3, Chapter A5, 29 p.

Lamke, A. D., 1969, Stage-discharge relations on Big Lost River within National Reactor Testing Station, Idaho: U.S. Geological Survey Open-File Report, IDO-22050, 28 p.

McKinney, J. D., 1985, Big Lost River 1983-1984 flood threat: U.S. Department of Energy, Idaho Operations Office Publication, PPD-FPB-002, 43 p.

Shearman, J. O. Kirby, W. H., Schneider, V. R., and Flippo, H. N., 1986, Bridge Waterways Analysis Model/Research Report; Federal Highway Administration report, FHWA-RD-86/108, 112 p.