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COOLING-WATER-INTAKE AND BLOWDOWN-DISCHARGE STUDY FOR CARROLL COUNTY POWER STATION

by

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PART ONE: Phase I. Bathymetric, Topographic, and Hydrometric Surveys

PART TWO: Phase II: Preliminary Sizing of Principal Components of Intake and Discharge Structures

PART THREE: Phase III. Laboratory Model Investigation of the Flow Pattern around Proposed Intake and Discharge Structures

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PART ONE

PHASE I. BATHYMETRIC, TOPOGRAPHIC, AND HYDROMETRIC SURVEYS

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ABSTRACT

Bathymetric, topographic, and hydrometric surveys have been carried out on the Mississippi River between River Miles 530 and 531. The data have been analyzed statistically in order to establish a hydraulic basis for a decision on the siting and general layout of the intake and discharge structures for the Carroll County Power Station. The results are presented in terms of N-day low values of discharge for various parts of the cross-section of the river.

For a nearshore channel running along the east bank of the river the 7-day low values are estimated to 1300, 1100, and 1000 cfs for recurrence intervals of 2, 5, and 10 years, respectively. Approximately 40% of the discharge in the nearshore channel originates from a breach in the Spring Lake Dike 4000 ft north of the study area.

Provided that the present river bed profile is maintained the nearshore channel can provide the necessary flow of cooling water for the power plant.

The question of whether the nearshore channel suffices as a receiver of blowdown discharge from the power station will be investigated in Phase II of this study.

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COOLING-WATER-INTAKE AND BLOWDOWN-DISCHARGE STUDY
FOR CARROLL COUNTY POWER STATION. PHASE I

I. INTRODUCTION

This report presents the results of Phase I of the cooling-water intake and blowdown-discharge study for Carroll County Power Plant. The investigation was commissioned by Commonwealth Edison Company, Chicago, Illinois, and is being carried out in accordance with proposal submitted 10 October 1977.

The purpose of the study is to establish a basis for the design of intake and discharge structures such that

- the structures will have good hydraulic performance;
- the structures will require minimum maintenance;
- thermal and other environmental standards will be met.

The intake and discharge structures are to be located on the Mississippi River at River Mile 530, slightly north of Thomson, Illinois. The anticipated maximum intake rate is between 70 and 95 cfs. Conservatively, 100 cfs was used in this study. The anticipated maximum blowdown is between 10 and 30 cfs. Conservatively, 60 cfs was used for this study.

Phase I includes bathymetric, topographic, and hydrometric surveys; and statisical analysis of river discharges. The results provide the hydraulic basis for a decision on the siting and general layout of the intake and discharge structures.

II. FIELD SURVEYS

The area surveyed is shown in figure 1. The area extends from the east bank of the Mississippi River to the west side of the navigation channel between River Miles 529 and 531.

The first survey was carried out during 24-26 July 1974 and included measurements of depth, current velocity, and visibility, together with observations of bottom sediment type and benthic fauna. Measurements and

samples were taken along a number of transects spaced 500 ft apart and located as shown in figure 1. Results of this survey were reported by Tatinclaux (1974).

In the 1974 survey only limited data, sufficient to define the principal features of the area, were collected. Additional surveys were carried out in 1977, from 4 September to 1 November. The purpose of these surveys were to verify and supplement the data collected in the 1974 survey. The 1977 surveys included profiling of the river bed along Sections 1 through 10 (see figure 1). Sonic sounding equipment, sounding lines, and laser distance-measuring apparatus were used. The results of bathymetric measurements are shown in figures 2 and 3. Along Sections 5, 7 and 10 velocity distributions were measured using an Ott propeller current meter. In addition, data (depth and velocity distribution) were collected at the breach in the Spring Lake Dike, 4000 ft north of Section 1, in order to determine the significance of the dike breach to the flow in the near-snore channel. Data also were collected in the two cable channels between Sections 7 and 9.

The results of the velocity measurements are shown in table 1 and figure 4. The mean velocities were calculated from three-point field data from the verticals where the velocity measurements were made; each mean velocity so determined the average velocity for the section extending between midpoints between adjacent measurement sections.

The velocity measurements were made during two survey series: 10-13 October, and 1 November 1977. In both series velocities were measured in the nearshore channel at Section 7 (see table 1), and the discharges through the channel were calculated: On 1 November the discharge was 2530 cfs, and during 10-13 October it was 3000 cfs. Knowing this difference, the measurements from both survey series could be reduced to a common base, which was taken to be the average discharge for the two series. The discharges measured on 1 November were then increased by 9 per cent while the discharges measured during 10-13 October were reduced by 8 per cent. The resulting flow pattern is shown in figure 5.

As seen in figure 5, the total discharge passing through the study area was about 40,000 cfs (the average over 10-13 October, and 1 November 1977). Only 10 per cent (4,000 cfs) of this passed over the slough areas between the river bank and the dredge-spoil islands. The flow through the nearshore channel

at the east bank was approximately 3,000 cfs, of which 1,200 cfs originated from the breach in the Spring Lake Dike.

The calculated value of the total discharge (40,000 cfs) is in reasonable agreement with the values recorded at Clinton, Iowa, by U.S. Geological Survey: On 12 October the discharge was 44,400 cfs, and on 1 November it was 36,300 cfs. (These values are preliminary, and subject to adjustment by the U.S.G.S.)

III. FLOW STATISTICS

To be of value in design considerations, the flow pattern depicted in figure 5 has to be related to reliable flow statistics, and the flow distribution for other river discharges projected. It is of particular interest to ascertain if the discharge in the nearshore channel is adequate for the intake-discharge structures. To this end, the flow statistics for Section No. 7 were established. The analysis is based on Section 7 idealized as shown in figure 6. Areas 1 and 2 of the section represent the main channel; area 4 represents the nearshore channel; and area 3 the slough area between the nearshore channel and the dredge spoil island at the east side of the main channel. Relevant dimensions are listed in table 2, together with the discharges measured in the field surveys on 10-13 October and 1 November 1977.

The statistical analysis is based on discharge and stage records for pool 13 of the Mississippi River. Pool 13 extends from Dam No. 12 at Bellevue, Iowa, to Dam No. 13 at Clinton, Iowa. River discharge is recorded at U.S. Geological Survey Station No. 05420500, at Clinton, and processed by the U.S.G.S. office in Iowa City (Ref: Mr. Ivan Burmeister). Results obtained from the U.S.G.S. which are relevant to the present study are summarized in tables 3 and 4, and in figures 7 and 8.

Table 3 gives the N-day-average low flows with recurrence intervals of 5, 10, 20, 50, and 100 years. For example, the 7-day, 10-year low flow is estimated to be 14,360 cfs. The values are based on observations over the period 1941-1976 only; that is, the period after the construction of the present lock-and-dam system, and completion of the principal dams on

tributaries. Figure 7 shows the 7-day, low value distribution in detail. Figure 7 also shows the distribution based on the entire period of observation, 1878-1976. It is obvious that the difference between the two distributions is significant, and that discharge records from before 1941 should be disregarded. Table 4 and figure 8 show the estimate of the lowest mean value for 7 consecutive days of the spawning season, taken to be the months of April and May.

The stage in Pool 13 is maintained at or above elevation 582 ft (above MSL) to insure adequate depth for barge navigation at all discharges. Stage records were obtained from the U.S. Army Corps of Engineers, Rock Island, Illinois. The records include daily gage heights for Gage Station SABULA, which is slightly north of the study area. The variations in stage at Sabula and in the study area are practically identical. Results of a statistical analysis of the stage data are shown in figure 9. The analysis covers a period of 10 years from 1967 to 1976.

It is seen in figure 9 that 50 per cent of the gage heights at Sabula are greater than 11.30 ft (Gage zero is 572.27 ft above MSL), and 90 per cent are greater than 10.70 ft.

The stage value on 1 November, and on 12 October 1977 (the days velocities were measured along Section 7) was 11.30. ft. Therefore, the one-day 10-year low value (which is slightly less than the 7-day 10-year low value) is only approximately 0.6 ft below the value on the days of the field surveys. A decrease of that amount has little effect on the relative distribution of discharge in the various parts of the cross-section. Estimates of 7-day low values of discharge through the four parts of cross-section are shown in table 5 and 6. The values in table 5 are based on records for the whole year while the values in table 6 are based on records for the months of April and May. The estimates are conservative since they are made under the assumption that the low values of the stage is fully correlated to the low values of the discharge. Due to the stage control in the area, the correlation between stage and discharge is not complete.

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IV. DISCUSSION

It has been suggested that decisions on the siting and layout of the intake and discharge structures be based on 7-day 10-year low flow values.

From table 5 it is seen that the 7-day 10-year low flow value for the near-shore channel is expected to be approximately 1000 cfs and greater than the anticipated rate of intake.
That is, if the 7-day 10-year low value is to be regarded as the design flow, the nearshore channel can provide the necessary flow of cooling water for the power plant. The intake therefore can be located at the nearshore channel, unless biological considerations indicate otherwise.

The discharge structure may also be located at the nearshore channel provided that thermal and other environmental standards are met. At a 7-day 10-year low flow in the nearshore channel it should be possible to obtain an initial dilution of the blowdown discharge of the order of 10. In the spawning season (April and May) there is enough discharge in the nearshore channel to dilute the blowdown discharge at least 20 to 30 times. The question of whether these dilution figures suffice from a biological point of view is beyond the scope of this report. Questions about dilution requirements will be investigated under Phase II of this study.

Approximately 40 per cent of the discharge in the nearshore channel originates from the breach in the Spring Lake Dike. If this breach is closed some time in the future, which has been considered, the velocity (and hence the discharge) in the nearshore channel is likely to be reduced. The channel may then be partly filled due to sedimentation.

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REFERENCE

Tatinclaux, J.C., "Bathymetric Survey of the Mississippi River in the Vicinity of the Proposed Location of the Blowdown Discharge Structure of the Carroll County Power Station", IIHR Limited Distribution Report No. 27, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, Iowa, September 1974.

TABLES

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Date	Location	Distance from shore (ft)	Water depth (ft)	Average velocity (fps)
10/10/77	Sect. 10	25	8.0	0.65
"	"	70	11.7	1.10
"	"	96	11.2	1.26
"	"	140	13.0	1.24
"	"	178	14.2	0.98
"	"	219	14.1	0.69
"	"	242	8.4	0.50
10/12/77	Sect. 8A-7A	180(1)	3.1	0.39(2)
"	"	224(1)	12.0	0.52(2)
"	"	260(1)	10.0	0.44(2)
"	"	280(1)	7.0	0.0
"	Sect. 7	60	7.0	0.70
"	"	88	10.0	0.82
"	"	136	12.0	0.70
"	"	157	13.0	0.71
"	"	181	14.5	0.85
"	"	243	7.5	0.80
"	"	286	6.0	0.67
"	"	337	7.0	0.58
"	"	398	5.2	0.34
"	"	445	5.2	0.42
"	"	493	5.0	0.55
"	"	538	4.7	0.52
"	"	613	4.2	0.46
"	"	668	4.0	0.45
"	"	729	4.0	0.38
"	"	776	3.7	0.41
"	"	855	2.5	0.27
"	"	1251	3.1	0.42
"	"	1330	3.6	0.45
"	"	1395	3.3	0.39
"	"	1444	2.3	0.48
"	Sect. 8A-9B	429(3)	9.1	0.13(4)
"	"	455(3)	8.5	0.16(4)
10/13/77	Dike breach	6(5)	6.1	0.55
"	(total width	28(5)	22.0	0.17
"	= 215 ft)	60(5)	31.0	0.16
"	"	113(5)	30.6	0.46
"	"	157(5)	17.5	0.22

(1) distance from point 8A towards 7A
(2) flow going west
(3) distance from point 8A towards 9B
(4) flow going east
(5) distance from west bank of opening

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Table 1. Velocity Measurements, 1977

Date	Location	Distance from shore (ft)	Water depth (ft)	Average velocity (fps)
10/13/77	Sect. 5	40	3.0	0.38
"	"	88	8.4	0.78
"	"	128	9.5	0.91
"	"	210	10.9	0.84
"	"	170	10.2	0.89
"	"	291	10.5	0.77
"	"	345	9.2	0.68
"	"	402	8.8	0.57
"	"	476	6.8	0.45
"	"	528	6.1	0.47
"	"	595	5.7	0.48
"	"	668	5.1	0.42
"	"	747	3.8	0.17
"	"	1198	3.9	0.31
"	"	1251	3.2	0.22
"	Sect. 10	96(6)	4.2	0.0
"	"	121(6)	6.0	0.18
"	"	146(6)	5.8	0.64
"	"	162(6)	5.3	0.64
"	"	178(6)	3.5	0.77
"	"	210(6)	3.2	0.71
"	"	277(6)	3.8	0.58
"	"	330(6)	2.3	0.49
11/1/77	Sect. 7	113	12.0	0.53
"	"	158	14.0	0.60
"	"	618	4.1	0.43
"	"	1400	3.4	0.37
"	"	208(7)	18.0	0.35
"	"	311(7)	18.0	0.97
"	"	400(7)	19.1	0.97
"	"	662(7)	15.1	0.60
"	"	946(7)	16.4	1.45
"	"	1301(7)	15.8	1.89
"	"	1508(7)	12.9	1.57
"	"	1894(7)	9.0	1.23
"	Sect. 5	208	11.0	0.76
"	"	2131	3.3	0.34
"	"	2298	12.0	0.47
"	"	2452	14.7	0.83
"	"	2685	10.0	0.74
"	"	2990	8.7	0.30
"	"	3444	19.4	1.40

(6) distance from 10B
(7) distance from 7B

Table 1. Velocity Measurements, 1977 (cont.)

Area No.	Width (ft)	Mean Depth (ft)	Discharge (cfs)	Discharge (% of total)
1	1600	12.9	29300	73.3
2	500	17.9	6700	16.7
3	1000	3.0	1200	3.0
4	500	8.6	2800	7.0

Table 2. Subdivision of Cross-Section No. 7. Depths and Discharges as Measured October 12 and November 1, 1977.

Number of Consecutive Days, N	Recurrence Interval				
	100	50	20	10	5
1	10370	11120	12280	13350	14680
3	10940	11660	12780	13820	15130
7	11630	12300	13360	14360	15650
14	12000	12720	13860	14960	16380
30	13410	14020	15040	16070	17480
60	14270	14940	16080	17240	18870
90	14750	15580	16960	18350	20260
120	15810	16570	17880	19250	21200
183	15380	16440	18220	20050	22630
365	23700	25840	29300	32630	37000

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Table 3. Estimate of N-day Low Values of River Discharge at Various Return Periods.

Number of Consecutive Days, N	Recurrence Interval				
	100	50	20	10	5
7	17240	19550	23490	27530	33180

Table 4. Estimate of the 7-day Low Value of the River Discharge at Various Return Periods; for the Months of April and May

Area No.	Width (ft)	Mean Depth (ft)	Discharge (cfs)	Discharge (% of total)	Recurrence Interval
1	1600	12.9	13470	73.3	2
		12.4	11520	73.6	5
		12.3	10580	73.7	10
2	500	17.9	3070	16.7	2
		17.4	2660	17.0	5
		17.3	2440	17.0	10
3	1000	3.0	550	3.0	2
		2.5	410	2.6	5
		2.4	360	2.5	10
4	500	8.6	1290	7.0	2
		8.1	1060	6.8	5
		8.0	980	6.8	10

Table 5. Subdivision of Cross-Section No. 7. Estimates of 7-day Low Values of Depths and Discharges for Recurrence Intervals of 2, 5, and 10 years.

Recurrence Interval	Area 1	Area 2	Area 3	Area 4
2	34200	7790	1400	3270
5	24320	5540	1000	2320
10	20180	4600	830	1930

Table 6. Subdivision of Cross-Section No. 7. Estimates of 7-day Low Values of Discharge (cfs) for April and May.

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FIGURES

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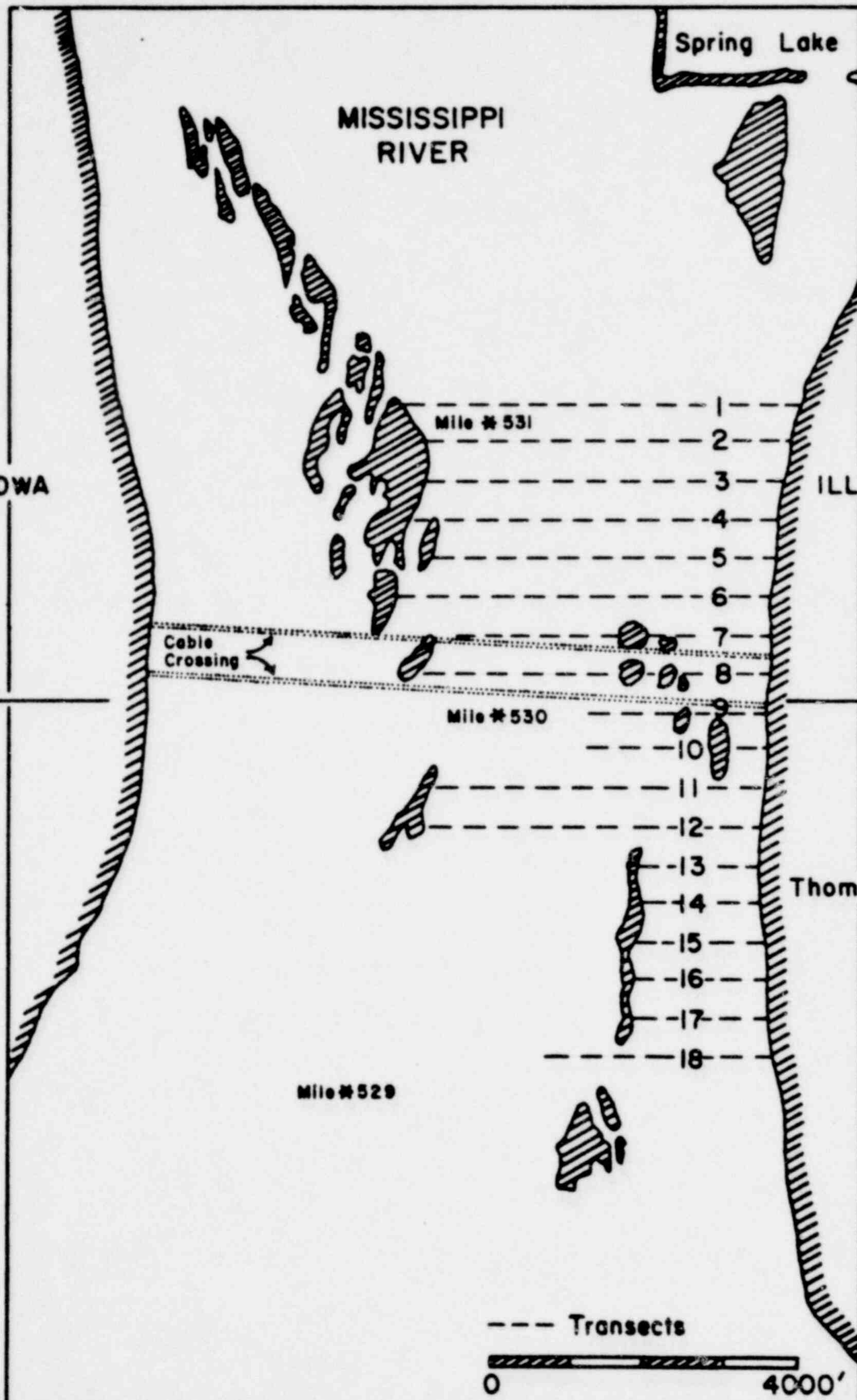

Figure 1 Study Area

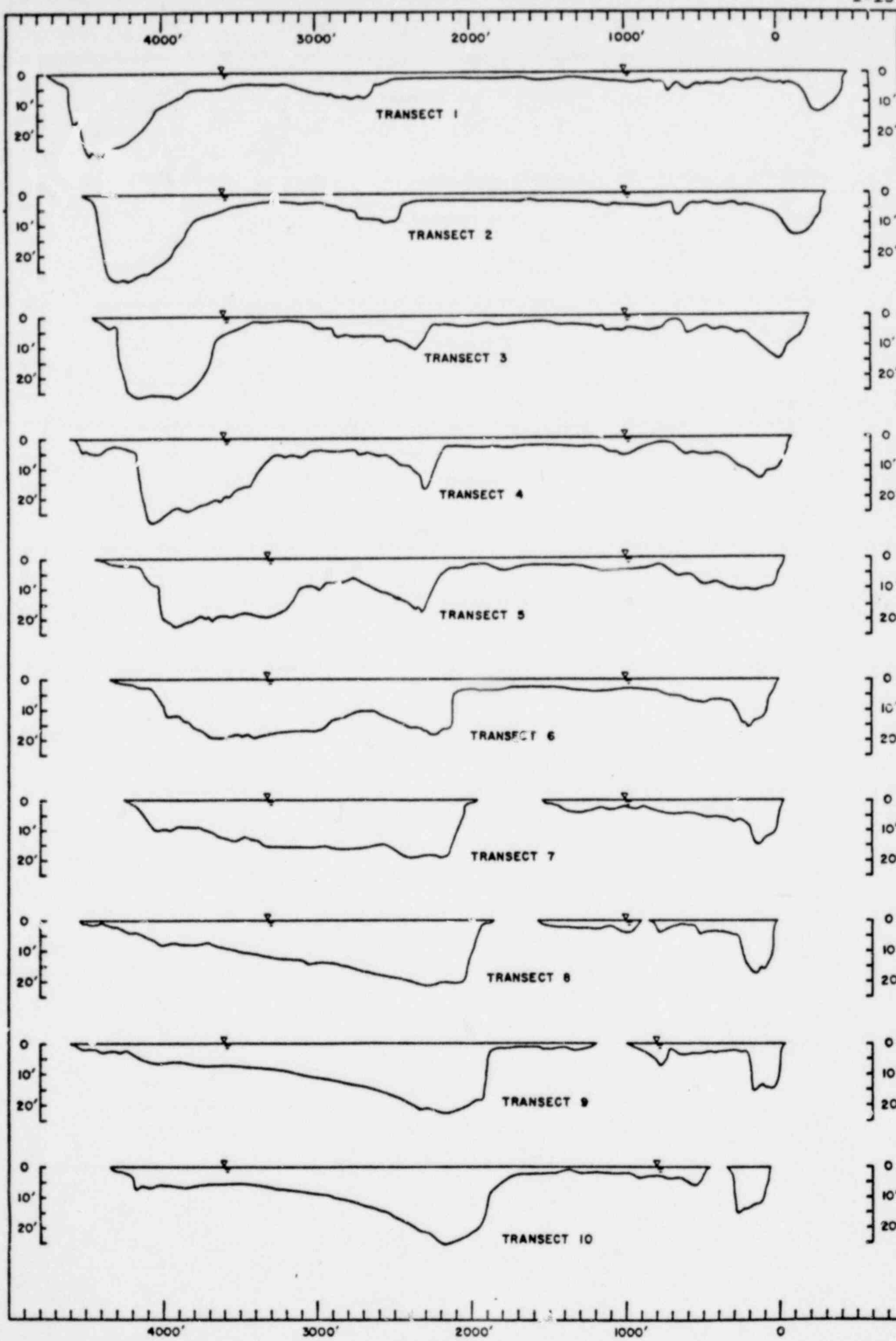

Figure 2 Transverse Bed Profiles

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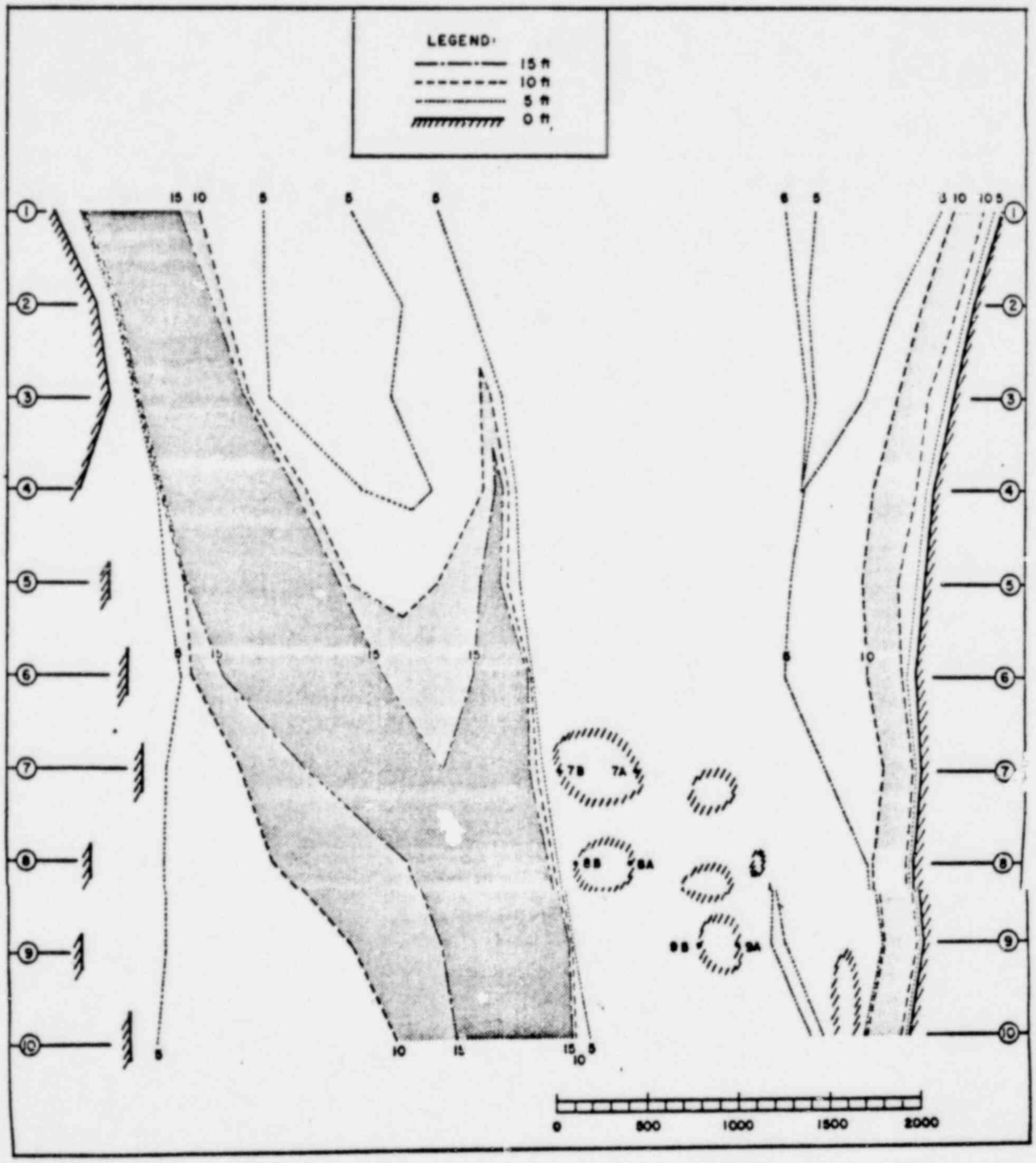

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Figure 3 Topography of Study Area

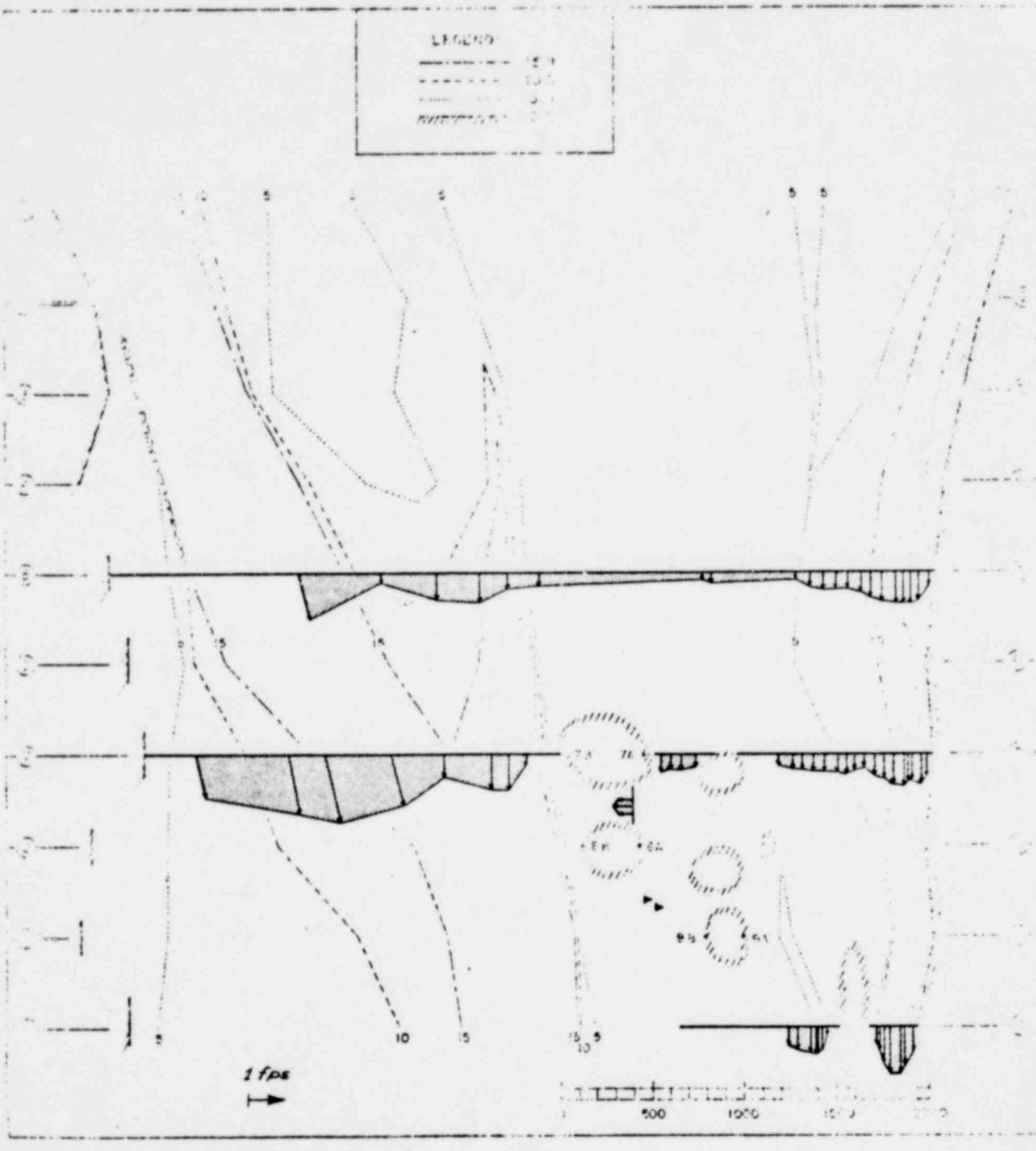

Figure 4 Depth-Averaged Distributions
of Current Velocity

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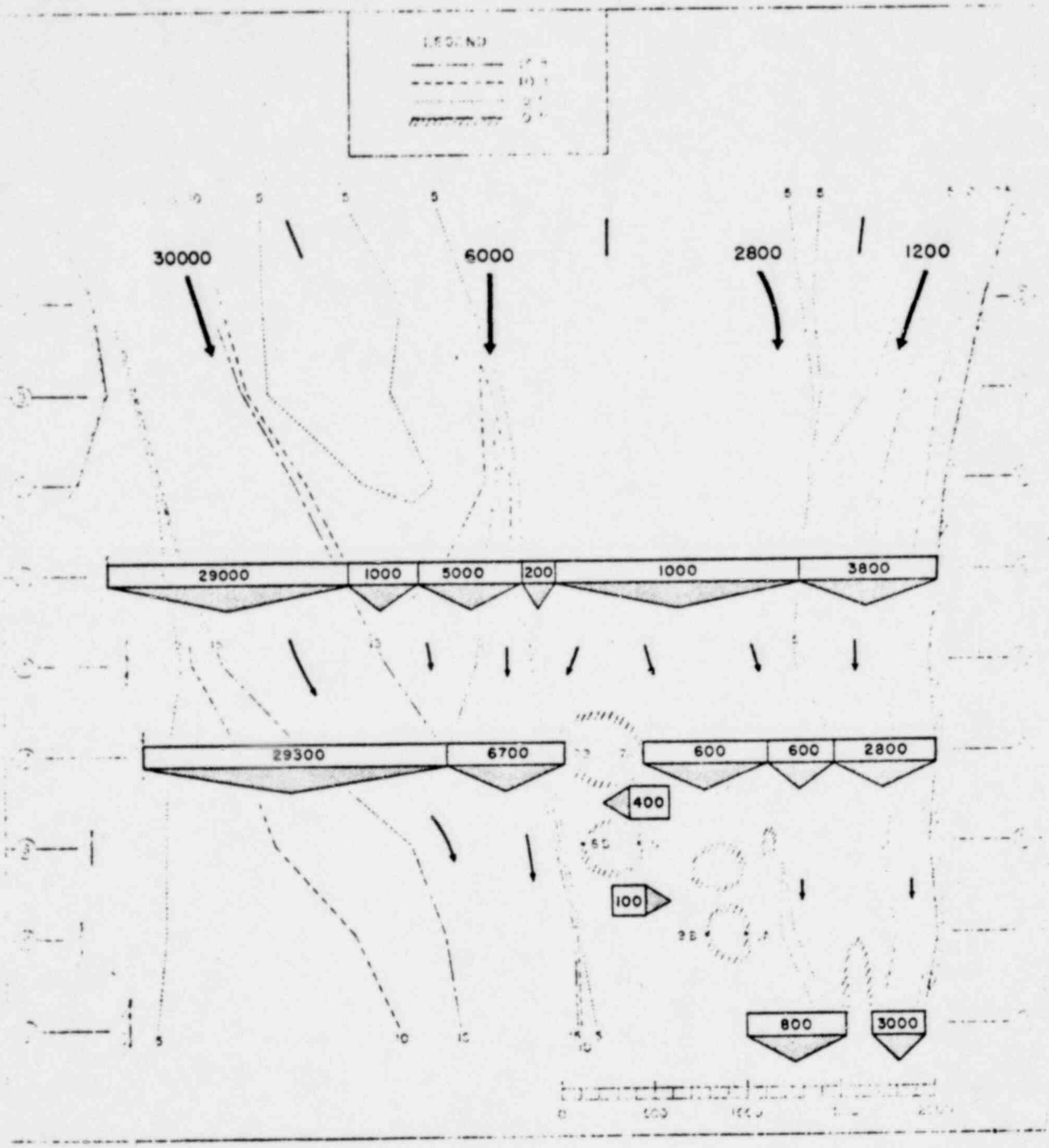

Figure 5 General Flow Pattern of Study Area. Figures are in Cubic Feet per Second

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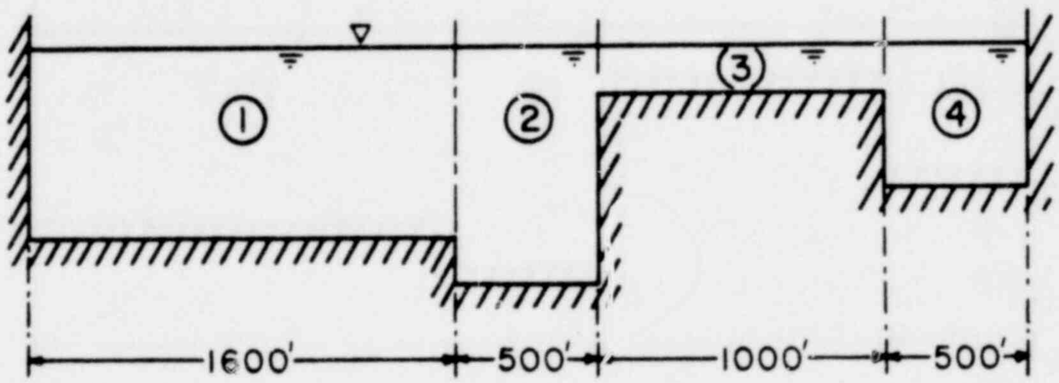

Figure 6 Idealized Cross-Section at Transect 7

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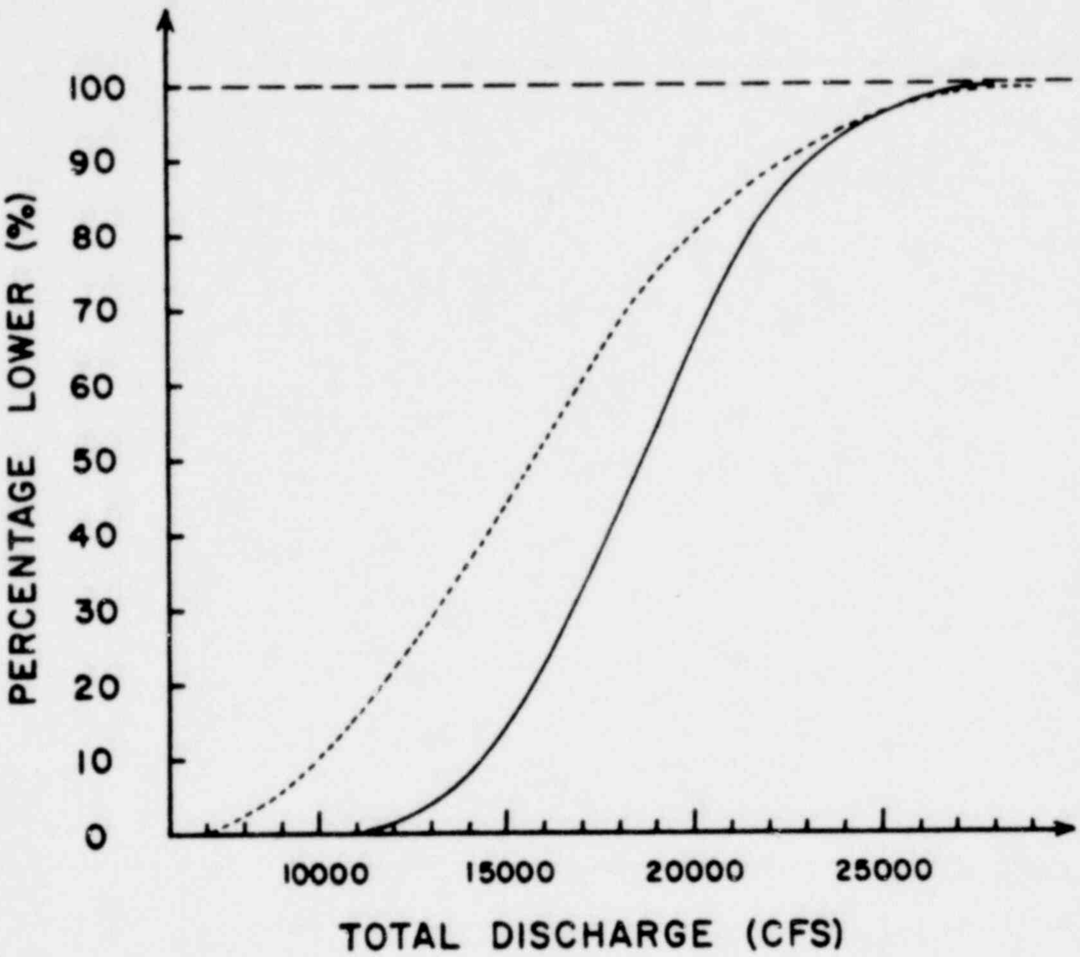

Figure 7 Cumulative Frequency Distribution for 7-day Low Value of discharge.
——— Based on Observations 1941-1976
----- Based on Observations 1878-1976

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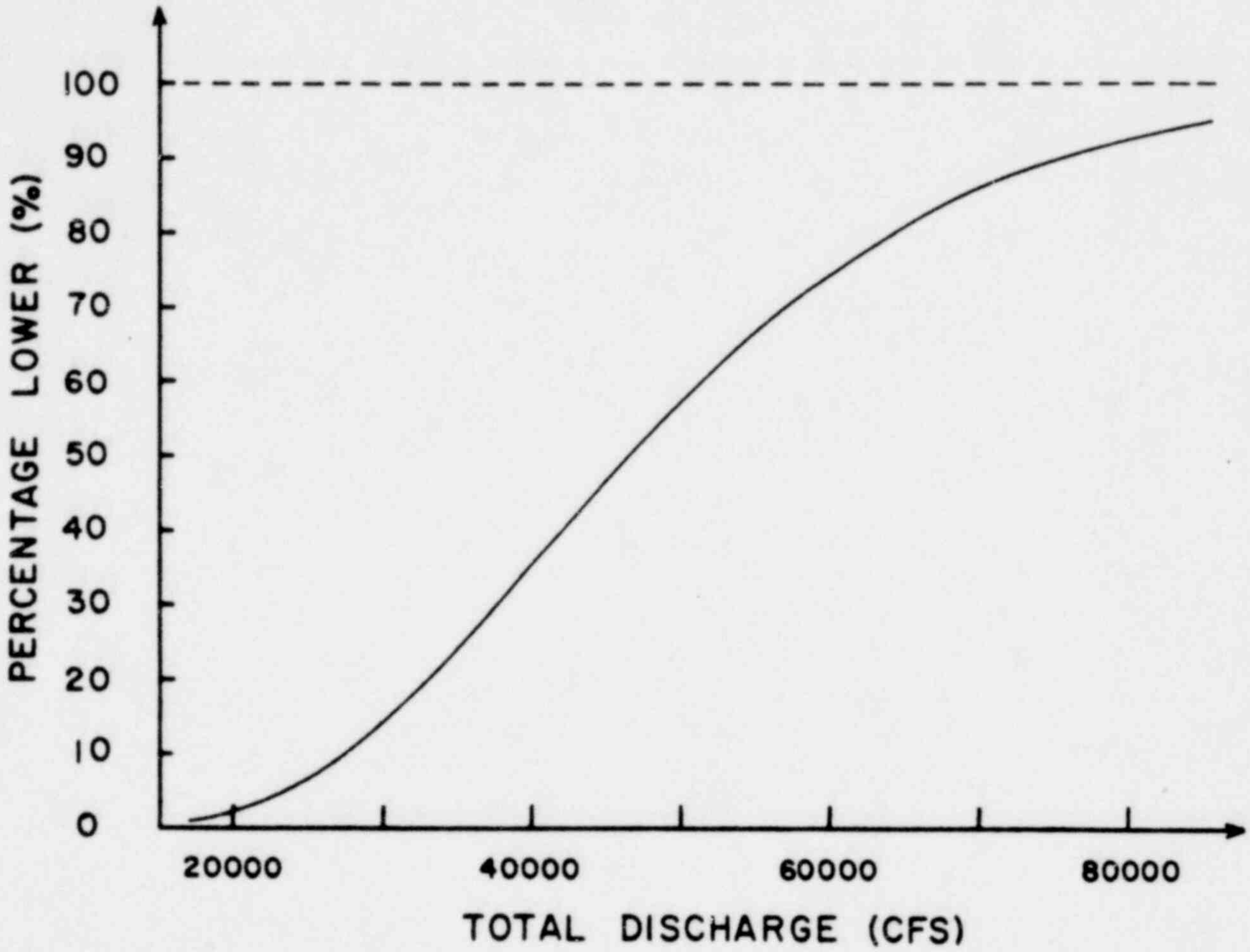

Figure 8 Cumulative Frequency Distribution for 7-day
Low Value of Discharge for April and May

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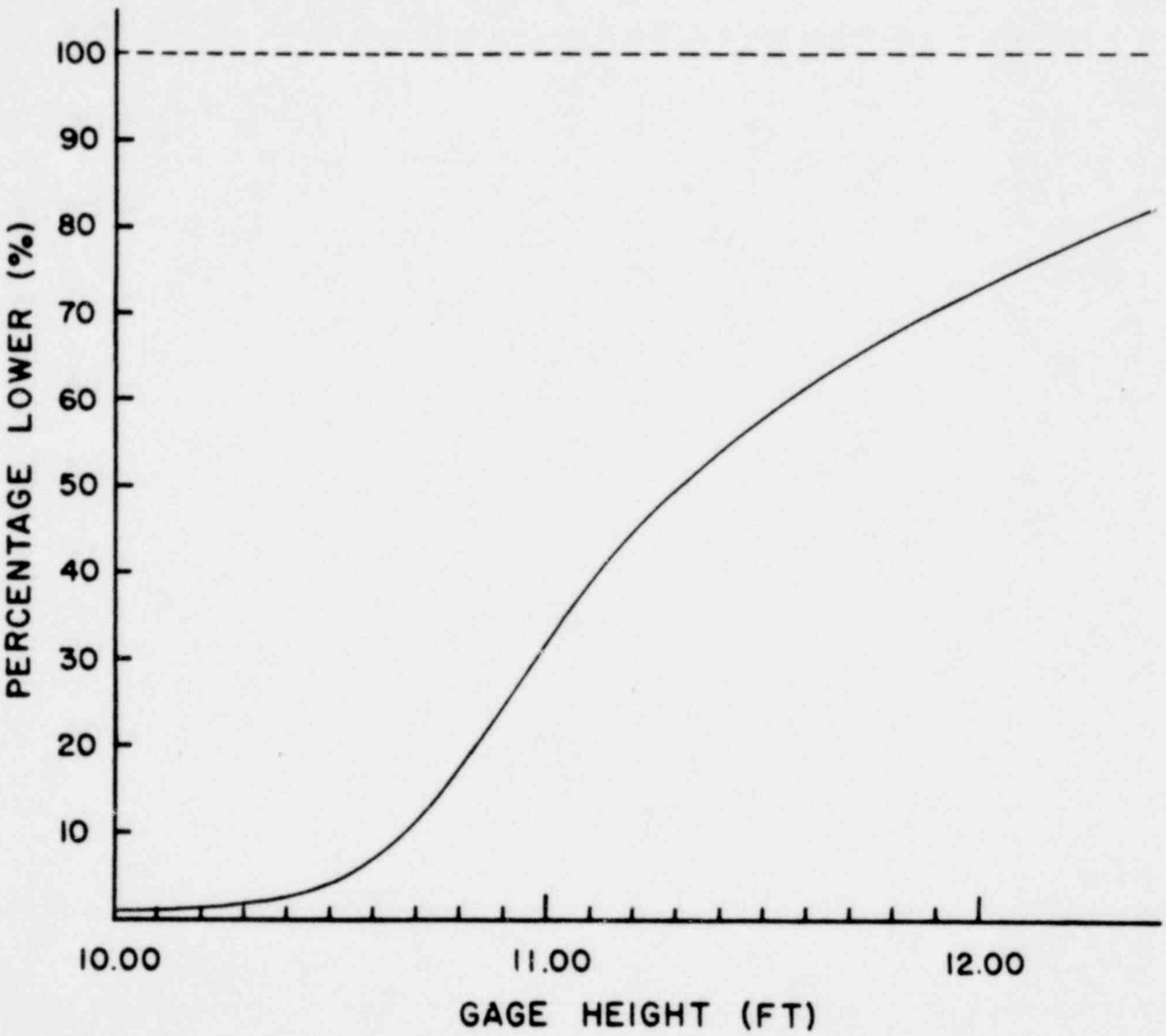

Figure 9 Cumulative Frequency Distribution for One-day Low Values of Stage at Sabula

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PART TWO

PHASE II. PRELIMINARY SIZING OF PRINCIPAL COMPONENTS
OF INTAKE AND DISCHARGE STRUCTURES

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ABSTRACT

A discussion and tentative sizing of the principal components of the cooling-water-intake and blowdown discharge structures for Carroll County Power Plant are presented together with a plan of study for the laboratory model investigation.

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COOLING-WATER-INTAKE AND BLOWDOWN-DISCHARGE STUDY
FOR CARROLL COUNTY POWER STATION. PHASE II

I. INTRODUCTION

This report presents the results of Phase II of the cooling-water-intake and blowdown-discharge study for Carroll County Power Station. The investigation was commissioned by Commonwealth Edison Company, Chicago, Illinois, and is being carried out in accordance with the proposal submitted to CECo by IIHR 10 October 1977.

The purpose of the study is to establish a basis for the design of intake and discharge structures such that

- the structures will have good hydraulic performance;
- the structures will require minimum maintenance;
- thermal and other environmental standards will be met.

The intake and discharge structures are to be located on the Mississippi River at River Mile 530, slightly north of Thomson, Illinois. The intake rate is expected to be up to 100 cfs, and the blowdown discharge up to 60 cfs.

Phase II includes

- hydraulic calculations adequate for a preliminary sizing of the principal components of the structures;
- plan of study for the laboratory model investigations.

II. INTAKE STRUCTURES

Three intake systems have been studied: A conventional shoreline intake with vertical traveling screens; perforated pipes in an infiltration gallery; and perforated pipes supported just above the river bed. The intake systems are to be designed on the basis of a maximum intake rate of 100 cfs with a depth-averaged approach velocity of less than 0.5 fps.

The intake system with perforated pipes in an infiltration gallery is not recommended. IIHR investigated the hydraulic aspects of this solution

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in an early stage of the Phase II study and presented the results in Draft 1 of the Phase II report. The remaining two intake systems are discussed in the following.

A. Conventional Shoreline Intake. Figures 1 and 2 show the shoreline intake considered. The layout is suggested by IIHR and is subject to revision by CECo. The dimensions shown are tentative and intended only to guide the engineers responsible for the detailed design of the intake.

The essential features of the structure are trash racks, traveling screens, and pumps. The trash racks form the face of the structure. They are aligned with the shoreline so that the inflow velocity through the screens is perpendicular to the river flow.

The dimensions of the intake face are chosen such that the depth-averaged inflow velocity is just under 0.5 fps at the minimum stage of the river (582 ft above MSL). The corresponding minimum depth in the intake has been chosen to be 8 ft, which is half the maximum depth in the nearshore channel. By continuity, the width of the intake face then has to be at least 25 ft.

As seen in figure 1, the intake can be considered as divided into 3 identical parts or "channels". In each channel there is a 33-inch, 50-cfs pump. One of the pumps serves as a standby. As indicated in figure 1, the width of each channel unit is (25/3) = 8.3.

Instead of three channel units with three 50-cfs pumps, four channel units with four 34-cfs pumps could be chosen.

The height of the intake structure is determined on the basis of stage records obtained from the U.S. Army Corps of Engineers, Rock Island, Illinois. The records include daily gage heights for Gage Station SABULA, which is slightly north of the study area. The variations in stage at Sabula and in the study area are practically identical. Results of a statistical analysis of the stage data are shown in figure 3. The analysis covers a period of 36 years from 1941 to 1976. It is seen in figure 3 that 1 per cent of the gage heights are greater than 16.3 ft (gage zero = 572.27 ft above MSL), and 0.1 per cent are greater than 20.5 ft. The maximum stage, 22.9 ft, was recorded on 27 April 1965.

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One of the major hydraulic problems is the design of the forebay. The distribution of the approach velocity over the trash racks depends on the flow pattern in the forebay which in turn depends on the combination of pumps operating.

B. Perforated Pipe Intake. The possibility of withdrawing the make-up water through a submerged perforated or slotted pipe was brought up in a meeting at CECo on 15 December 1977. The principal points discussed in this meeting are documented in a letter of 28 December 1977. The idea has been further discussed in a meeting on 1 February 1978 at the Johnson Division of UOP Inc., St. Paul, Minnesota. Conference notes have been prepared by Jacob Odgaard, IIHR, and distributed to conference participants from CECo and UOP.

A perforated or slotted pipe installed above the river bed is reportedly a very reliable intake system (Schreiber *et al.* 1974). The main design problem is to keep the approach velocity low and uniform over the screen surface. According to Mr. Lee Cook, UOP, in order to achieve this, the length of the intake pipe (screen) should be less than about twice the diameter of the pipe. For a diameter of 3 ft, a screen length of 7 ft is the maximum over which a reasonably uniform inflow velocity can be maintained.

Two alternative designs were suggested by Mr. Paul Fournier in the 1 February meeting at the Johnson Division. These are depicted in figure 4. Both alternatives are dimensioned on the basis of an open space area of 40 per cent and an approach velocity of 0.4 fps. Alternative A in figure 4 is designed for a flow rate of 10 cfs; alternative B for a flow rate of 20 cfs. Hence, to withdraw 100 cfs, ten alternative A units would be required, and five alternative B units.

There are, however, ways to reduce the number of intake units. Various alternatives are shown in figure 5. As seen in figure 5 (a), (b), and (c), one way is to vary the cross-sectional area of the screen; or, as shown in figure 5 (d), to place a cone shaped impermeable core inside the main intake pipe. Alternatively, the open space area could vary along the screen, as indicated in figure 5 (e). Another way would be to place another perforated pipe or screen inside the main pipe as shown in figure 5 (f) and (g). The open space area of the internal screen would have to be considerably

less than that of the outer screen. The internal screen could be cylindrical (figure 5 (f)) as suggested by Richards and Hroncich (1976) in which case its open space area would have to vary along the screen, or it could be cone shaped as indicated in figure 5 (g). The effect of these alternative designs would be to insure that the approach or inflow velocity be uniform over a greater screen length.

The cleaning of the screen is a fairly simple process. Backwashing by water is the normal procedure. However, according to Mr. Paul Fournier, UOP, backwashing by air should provide more effective cleaning. For the alternatives in figure 4 backwashing by air would require a 3- to 4-inch diameter pipe, a 2-HP pump, and a 40- to 50-gal tank.

An alternative or additional cleaning effect might be achieved by air bubbles produced from a pipe on the river bed beneath the screen. When sweeping around the screen, the bubbles would loosen and remove seaweed and debris stranded on the screens. Stopping the pumping is reportedly also an effective way to clean the screen surface.

The optimum orientation of the intake is probably one in which the screens are placed parallel rather than perpendicular to the direction of the river flow.

A pilot study of this intake system has been proposed. The suggested layout is shown in figures 6 and 7. The objectives of the study would be to test the functioning and performance of the system under the flow conditions in the nearshore channel. The test might start with an investigation of the performance of Alternative A in figure 4 and later extended to an investigation of the air backwashing technique and some of the alternatives of figure 5.

III. DISCHARGE STRUCTURE

The discharge of waste heat from the power station into the Mississippi River must comply with the thermal criteria of Illinois. The regulatory agency for the state is the Illinois Environmental Protection Agency. The applicable Illinois thermal criteria that apply to the Mississippi River, as compiled by Paily *et al.* (1976), are presented in the Appendix.

The most restrictive criterion appears to be that the maximum temperature rise above natural temperatures shall not exceed 5°F outside the mixing zone. As seen in the Appendix, the Illinois thermal standards specify "that no mixing zone shall exceed the area of a circle with a radius of 600 ft."

The blowdown discharge is expected to be up to 60 cfs, and its temperature can be as high as 24°F above the river temperature. In order to reduce the excess temperature to 5°F the heated discharge has to be diluted 4.8 times.

A preliminary estimate of the rate of dilution in the nearshore channel may be obtained on the basis of the results of the single-port prototype study at the Quad-Cities Nuclear Power Station by Parr and Sayre (1977). This prototype study consisted of seven surveys conducted between 11 November 1973 and 30 October 1974. The background data and results are presented in tables 5.8 and 5.9 in Parr and Sayre (1977). Figure 8 in the present report is prepared on the basis of these results. Figure 8 shows the minimum dilution, S, plotted as a function of a dimensionless downstream distance, x/D. x = distance downstream from the discharge port; D = port diameter. The minimum dilution, S, is defined as the ratio between the temperature rise across the condensers, ΔT_o, and the temperature rise above ambient temperature in the downstream jet, ΔT:

$$S = \Delta T_o/\Delta T$$

It is seen in figure 8 that a dilution of 4.8 at Quad-Cities was achieved at a downstream distance of approximately 20 times the port diameter. In the prototype survey the ratio, U_j/U_a, of the discharge velocity, U_j, to the ambient velocity, U_a, varied between 2.8 and 7.6. By varying this ratio over a wider range in a series of model tests, Parr and Sayre (1977) showed that the dilution at a given point downstream from the discharge port decreased for increasing values of U_j/U_a. The tests showed, however, that the decrease in dilution was not significant for

$$U_j/U_a \leq 30$$

$$x/D \leq 15$$

$$h/D \geq 7$$

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where h = depth of discharge. For $U_j/U_a \leq 30$, $h/D \geq 7$, and $x/D = 15$, Parr and Sayre measured a dilution of approximately 4.5.

At the 7-day, 10-year low flow in the nearshore channel, the average velocity at Transect No. 7 is estimated to be approximately 0.25 fps. The maximum water depth is about 15 ft. For U_a = 0.25 fps and h = 15 ft, the above mentioned constraints read

$$U_j \leq 7.5 \text{ fps}$$

$$x \leq 15 \text{ D}$$

$$D \leq 2 \text{ ft}$$

From these considerations it is reasonable to believe that a discharge structure can be designed such that the stipulated environmental criteria be met for a discharge into the nearshore channel. The number of ports in the discharge pipe woulc be one or two. In order to maintain a reasonably low value of the friction loss, the flow velocity in the discharge pipe should be less than 10 fps; that is, a 33-inch diameter discharge pipeline would be adequate.

IV. PLAN OF STUDY FOR LABORATORY MODEL INVESTIGATION

It is recommended that the area modeled be 1440 ft long and 1200 ft wide, and located as shown in figure 9. The southern boundary is 440 feet downstream from the proposed location of the pump house. Its western boundary runs through (and along) the inner dredge spoil islands. The discharge through the area will then be approximately 8 percent of the total river discharge (refer to Phase I report).

The discharges to be covered in the model investigation range from the 7-day 10-year low value to those characteristic of the springtime period of greatest biological sensitivity. For the part of the river to be modelled the 7-day 10-year low value is approximately 1150 cfs, and the 7-day 10-year low value for April and May is approximately 2200 cfs. However, it seems wise to choose a higher upper limit for the discharge to be modeled than 2200 cfs, since a higher discharge may give rise to significant changes in local flow patterns in the area. The fiftieth percentile of the monthly means for April, which is approximately 7300 cfs, would be a reasonable upper limit.

The main objective of the model tests will be to investigate the flow pattern around the intake and discharge structures for the above mentioned range of flow. The first type of intake to be investigated will be the conventional shoreline intake. The configuration of the intake will be one proposed by Sargent and Lundy, Chicago (their drawing no. CS-57). This intake is slightly different from the one described above in Chapter II, Section A. The discharge structure modeled will be the one described in Chapter III.

To properly simulate the kinematics and dynamics of the flow field, an undistorted model is recommended. Since inertial and gravitational forces are the dominant forces in the flow problem, model and prototype should have the same ratio of inertial forces to gravitational forces. This ratio is expressed by the Froude number

$$F = \frac{V}{\sqrt{gh}}$$

where

V = velocity

h = flow depth

g = acceleration of gravity

In a Froude model, the Froude number of the model,

$$F_m = \frac{V_m}{\sqrt{gh_m}}$$

is equal to the Froude number of the prototype,

$$F_p = \frac{V_p}{\sqrt{gh_p}}$$

Using $h_m/h_p = L_r$ where L_r is the length scale ratio,

$$V_r = \sqrt{L_r}$$

Subscript r denotes the ratio between model and prototype; i.e., $V_r = V_m/V_p$. The discharge ratio, Q_r, for the model to prototype is

$$Q_r = Q_m/Q_p = A_r V_r$$

where

$$A_r = L_r^2$$

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Hence,

$$Q_r = L_r^{5/2}$$

To insure a sufficient level of turbulence the Reynolds number,

$$Re_m = \frac{V_m h_m}{\nu}$$

(ν = kinematic viscosity) should be greater than 1500 in the nearshore channel. That means, the length scale ratio, L_r, should be less than 30. A value of 24 would be reasonable. This gives the following model characteristics:

Length of model:	60'
Width of model:	50'
Depth range:	1.5" to 7.5"
Average depth:	3"
Discharge range:	0.4 cfs to 2.6 cfs
Intake rates:	up to 0.035 cfs
Discharge rates:	up to 0.021 cfs
Width of intake:	26.5"

A plan view of the model layout is shown in figure 10.

The model will be calibrated against the results of the field measurements obtained on 10-13 October and 1 November 1977; that is, for a total river discharge of 40,000 cfs. The topography of the model will correspond to the topography measured in the field surveys conducted from 4 September to 1 November 1977, and in April 1978.

Discharges in the model will be measured by means of a calibrated orifice meter on the return pipe from the sump to the diffuser. Conventional point gauges will be used to measure water surface elevations. Velocities will be measured with a miniature current meter (propeller). Flow patterns will be visualized by means of floating confetti, injected dye and other flow visualization techniques.

The model tests will include measurements of the velocity distribution in the vicinity of the intake and discharge structures for various river discharges and for different combinations of intake pumps operating. The velocity distribution along the face of the intake structure will be measured in detail, and so will the velocity distribution immediately downstream from the discharge pipe.

The temperature of the discharge will be the same as that of the ambient water. It is, however, recommended that a separate heated-water

discharge study be carried out at a larger scale, say 1:10. The purpose of this study would be to optimize the discharge structure with respect to number and orientation of discharge ports and rate of temperature reduction.

V. CONCLUSION

A study of hydraulic aspects of a conventional shoreline intake and a submerged perforated pipe intake has been presented together with a tentative sizing of principal components of the structures; figures 1, 2 and 4.

The submerged perforated-pipe intake consists of perforated or slotted, 3-foot diameter screens placed 3 ft above the river bed. For an intake rate of 100 cfs, ten 7 feet long screen units are required (figure 4, alternative A) (or five 14 feet long screen units of the design shown in figure 4, alternative B). The number of screen units could be reduced by selecting one of the alternatives shown in figure 5.

A pilot study has been proposed to test the functioning and performance of the submerged perforated-pipe intake under the flow conditions in the nearshore channel.

The proposed discharge structure is a 33-inch diameter pipeline extending into the nearshore channel to a depth of 15 ft. The heated water is discharged through one or two diffuser ports.

Finally, a plan of study of laboratory model investigations has been presented. The model study is lelieved to make it possible to predict in detail the flow pattern around the shoreline intake and discharge structures. It is recommended that the area modeled be 1440 ft long and 1200 ft wide, and located as shown in figure 9.

REFERENCES

Paily, P.P., et al., October 1976. "Thermal Regimes of the Upper Mississippi and Missouri Rivers," IIHR Report No. 182, Iowa Institute of Hydraulic Research, The University of Iowa, Iowa City, Iowa.

Parr, A.D., and Sayre, W.W., June 1977. "Prototype and Model Studies of the Diffuser-Pipe System for Discharging Condenser Cooling Water at the Quad-Cities Nuclear Power Station", IIHR Report No. 204, Iowa Institute of Hydraulic Research, The University of Iowa, Iowa City, Iowa.

Richards, R.T., and Hroncich, M.J., February 1976. "Perforated-Pipe Water Intake for Fish Protection", Journal of the Hydraulics Division, ASCE, Vol. 102, No. HY2, p. 139.

Schreiber, D.L., Becker, C.D., Fuquay, J.F., and Chitwood, R.A., December 1974. "Intake System Assessment for Central Columbia River", Journal of the Power Division, ASCE, Vol. 100, No. PO2, p. 139.

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APPENDIX

REGULATIONS OF THE ILLINOIS ENVIRONMENTAL PROTECTION AGENCY CONCERNING THE DISCHARGE OF WASTE HEAT INTO THE MISSISSIPPI RIVER NEAR CLINTON, IOWA

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ILLINOIS

The Illinois Pollution Control Board Rules and Regulations (Chapter 3: Water Pollution) specify the temperature criteria for the Mississippi River as follows:

1. There shall be no abnormal temperature changes that may adversely affect aquatic life unless caused by natural conditions.
2. The normal daily and seasonal temperature fluctuations that existed before the addition of heat due to other than natural causes shall be maintained.
3. The maximum temperature rise above natural temperatures shall not exceed 5°F.
4. In addition, the water temperature at representative locations in the main river shall not exceed the maximum limits in the following table during more than one percent of the hours in the 12-month period ending with any month. Moreover, at no time shall the water temperature at such locations exceed the maximum limits in the following table by more than 3°F.

Zone 1: Mississippi River (Wisconsin Border to Iowa Border)

Zone 2: Mississippi River (Iowa Border to Alton Lock and Dam)

Zone 3: Mississippi River (South of Alton Lock and Dam)

Month	Zone 1 (°F)	Zone 2 (°F)	Zone 3 (°F)
January	45	45	50
February	45	45	50
March	57	57	60
April	68	68	70
May	78	78	80
June	85	86	87

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Month	Zone 1 (°F)	Zone 2 (°F)	Zone 3 (°F)
July	86	88	89
August	86	88	89
September	85	86	87
October	75	75	78
November	65	65	70
December	52	52	57

5. The owner or operator of a source of heated effluent which discharges 0.5 billion British thermal units per hour or more shall demonstrate in a hearing before this Board not less than 5 nor more than 6 years after the effective date of these regulations, or, in the case of new sources, after the commencement of operation, that discharges from that source have not caused and cannot be reasonably expected to cause significant ecological damage to the receiving waters. If such proof is not made to the satisfaction of the Board, appropriate corrective measures shall be ordered to be taken within a reasonable time as determined by the Board.

6. Permits for heated effluent discharges, whether issued by the Board or the Environmental Protection Agency, shall be subject to revision in the event that reasonable future development creates a need for reallocation of the assimilative capacity of the receiving stream as defined in the regulation above.

7. The owner or operator of a source of heated effluent shall maintain such records and conduct such studies of the effluents from such source and of their effects as may be required by the Environmental Protection Agency or in any permit granted under the Environmental Protection Act.

8. Appropriate corrective measures will be required if, upon complaint filed in accordance with Board rules, it is found at any time that any heated effluent causes significant ecological damage to the receiving stream.

Mixing Zones

a. In the application of any of the rules and regulations, whenever a water quality standard is more restrictive than its corresponding effluent standard, then an opportunity shall be allowed for the mixture of an effluent with its

receiving waters. Water quality standards must be met at every point outside of the mixing zone. The size of the mixing zone cannot be uniformly prescribed. The governing principle is that the proportion of any body of water or segment thereof within mixing zones must be quite small if the water quality standards are to have any meaning. This principle shall be applied on a case-by-case basis to ensure that neither any individual source nor the aggregate of sources shall cause excessive zones to exceed the standards. The water quality standards must be met in the bulk of the body of water, and no body of water may be used totally as a mixing zone for a single outfall or combination of outfalls. Moreover, except as otherwise provided, no single mixing zone shall exceed the area of a circle with a radius of 600 feet. Single sources of effluents which have more than one outfall shall be limited to a total mixing area no larger than that allowable if a single outfall were used.

In determining the size of the mixing zone for any discharge, the following must be considered:

1. The character of the body of water.

2. The present and anticipated future use of body of water.

3. The present and anticipated water quality of the body of water.

4. The effect of the discharge on the present and anticipated future water quality.

5. The dilution ratio.

6. The nature of the contaminant.

b. In addition to the above, the mixing zone shall be so designed as to assure a reasonable zone of passage for aquatic life in which the water quality standards are met. The mixing zone shall not intersect any area of any such waters in such a manner that the maintenance of aquatic life in the body of water as a whole would be adversely affected, nor shall any mixing zone contain more than 25% of the cross-sectional area or volume of flow of a stream except for those streams where the dilution ratio is less than 3:1.

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FIGURES

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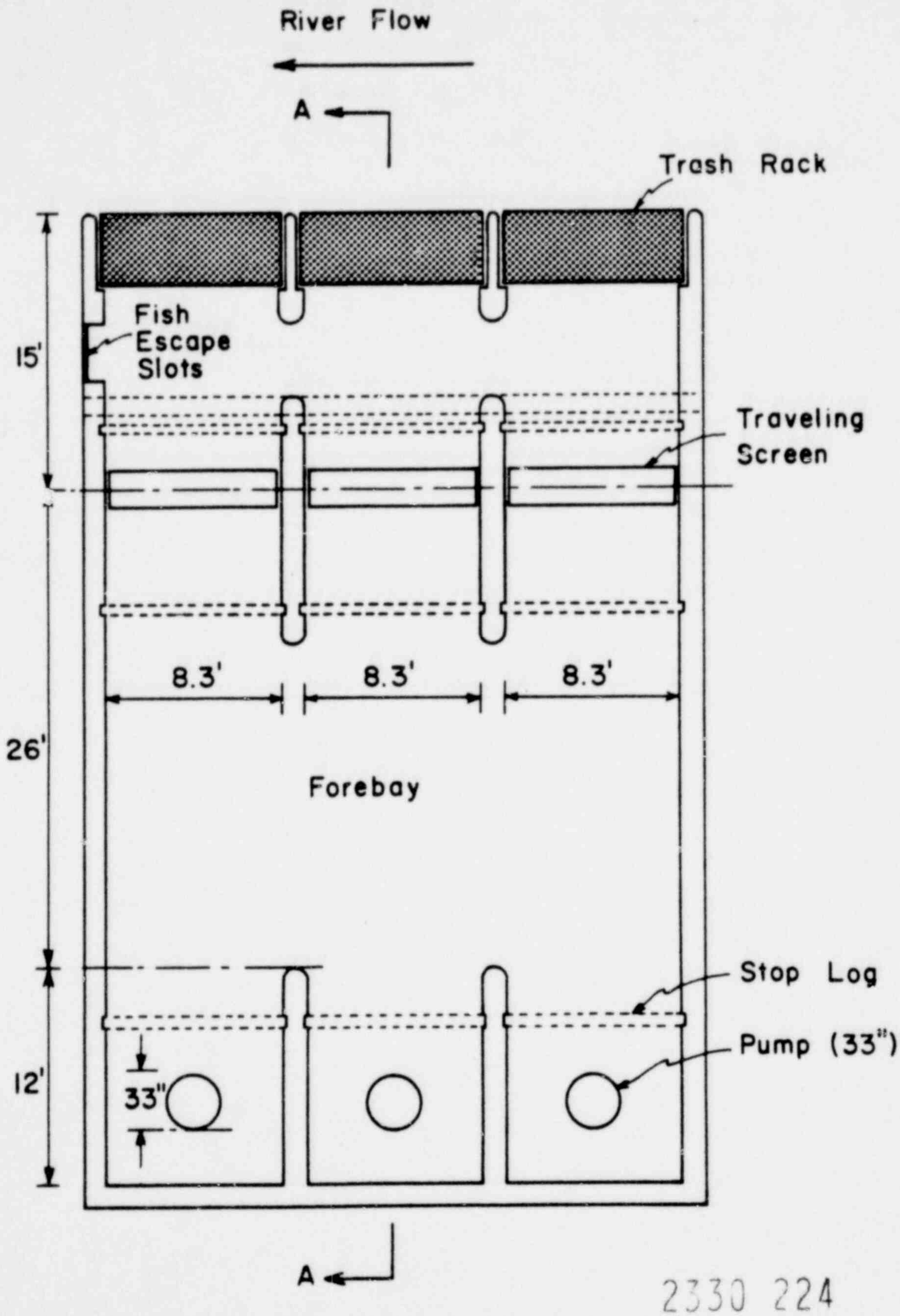

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Figure 1: Plan view of proposed 100 cfs shoreline intake

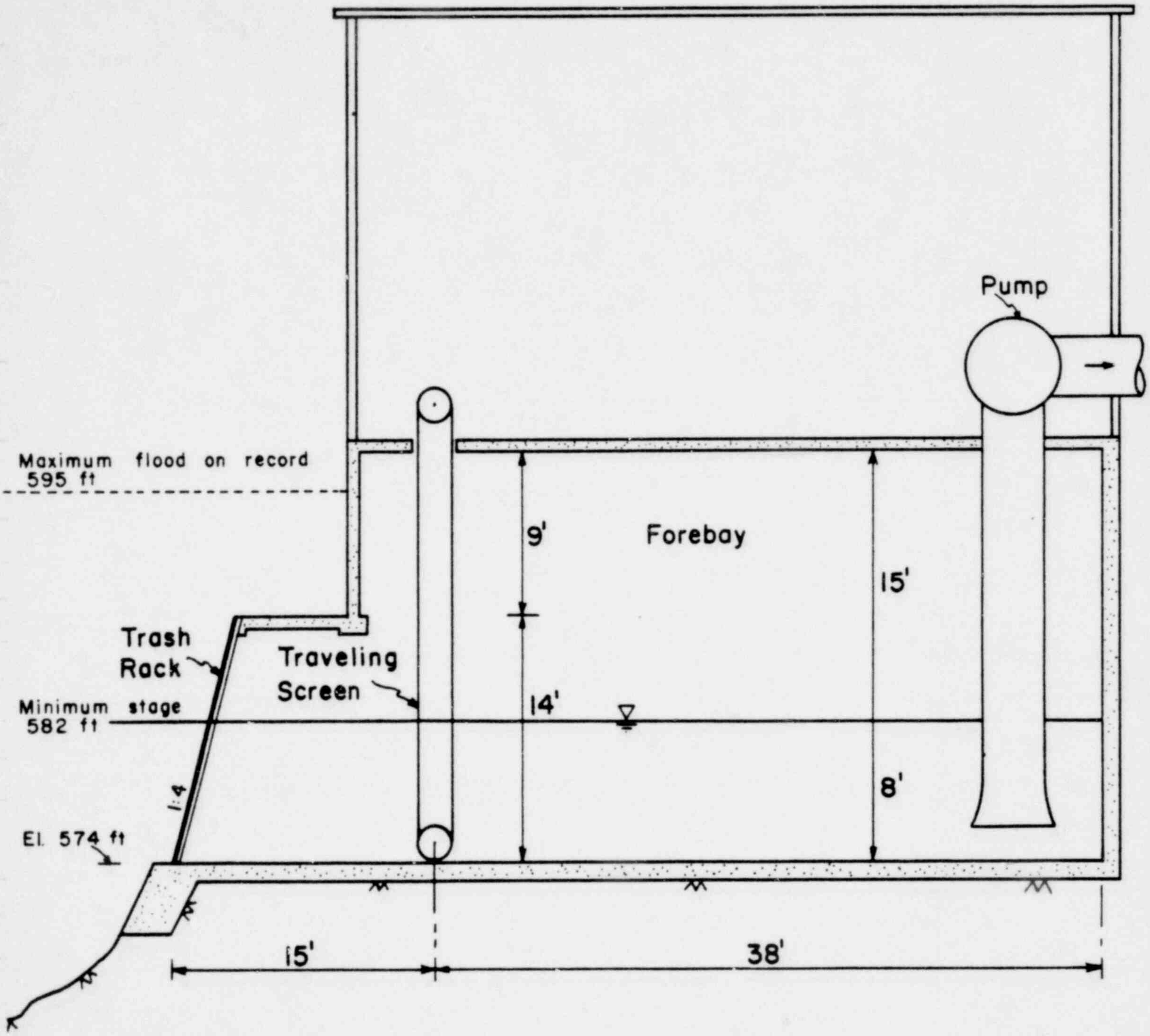

Figure 2: Section A-A of proposed shoreline intake

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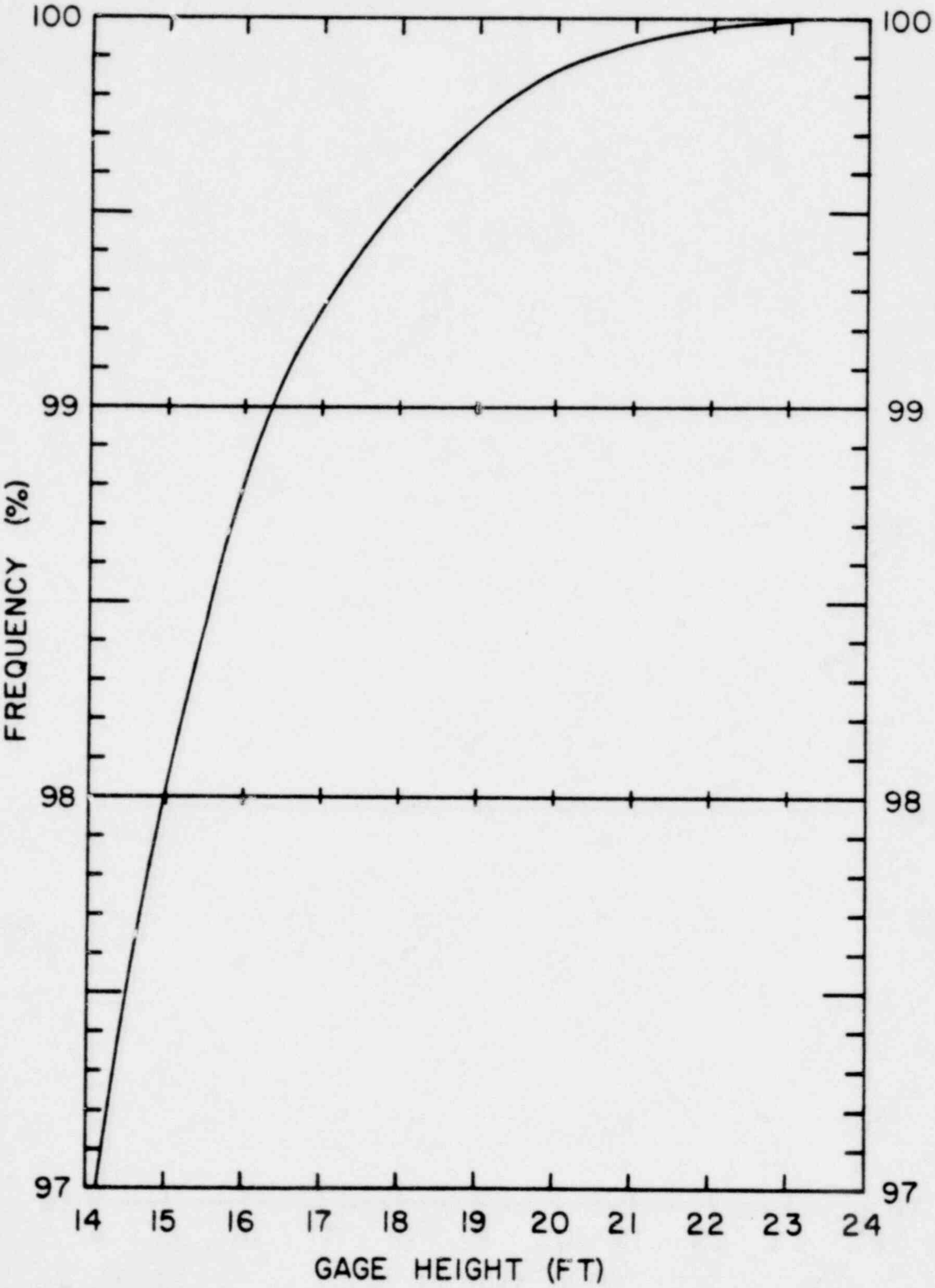

Figure 3: Cumulative frequency distribution for gage heights at Sabula. 1941-1976

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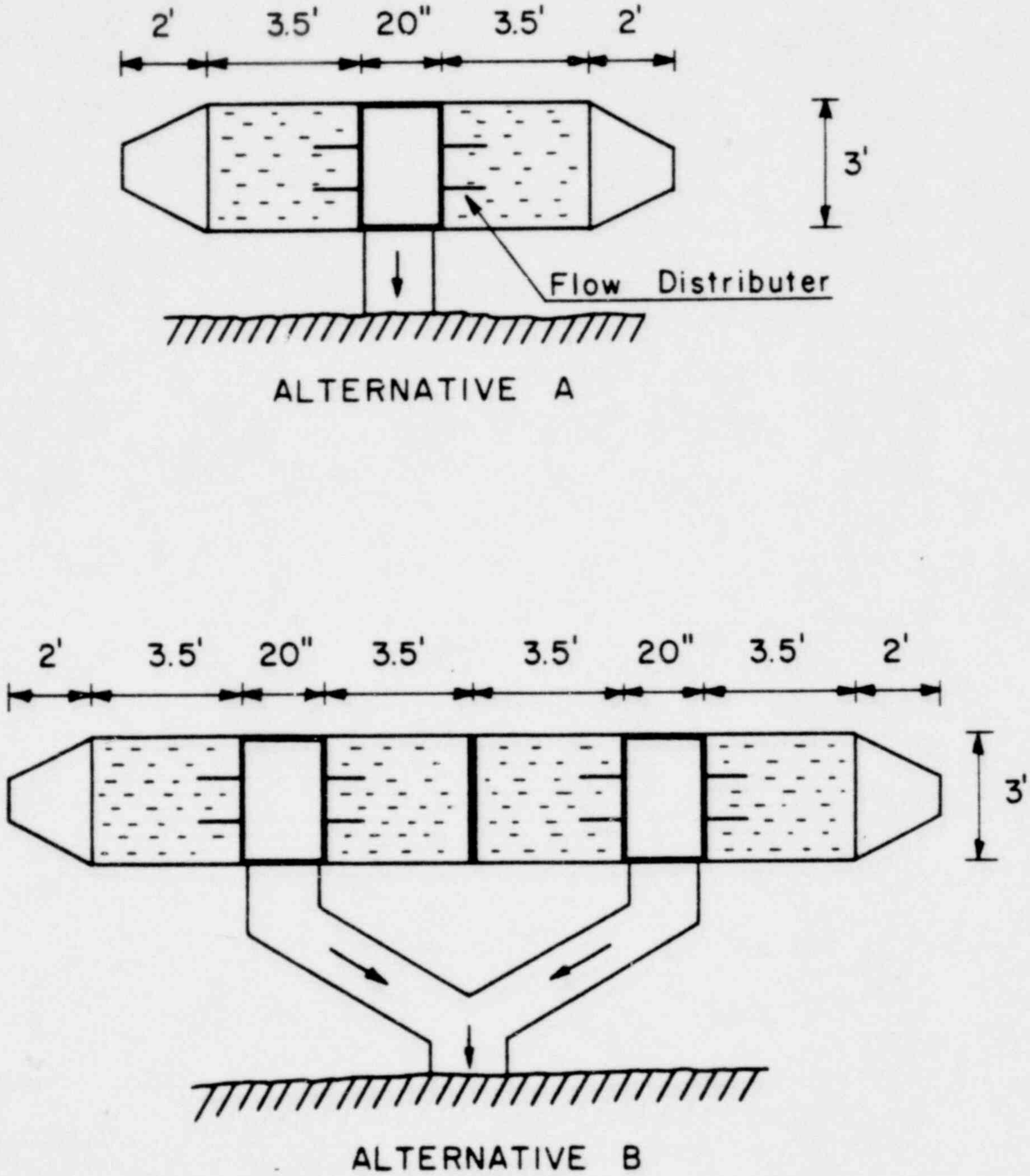

Figure 4: Two alternative designs of a perforated-pipe intake unit. Suggested by P. Fournier, Johnson Division, UOP, Inc., St. Paul, Mn.

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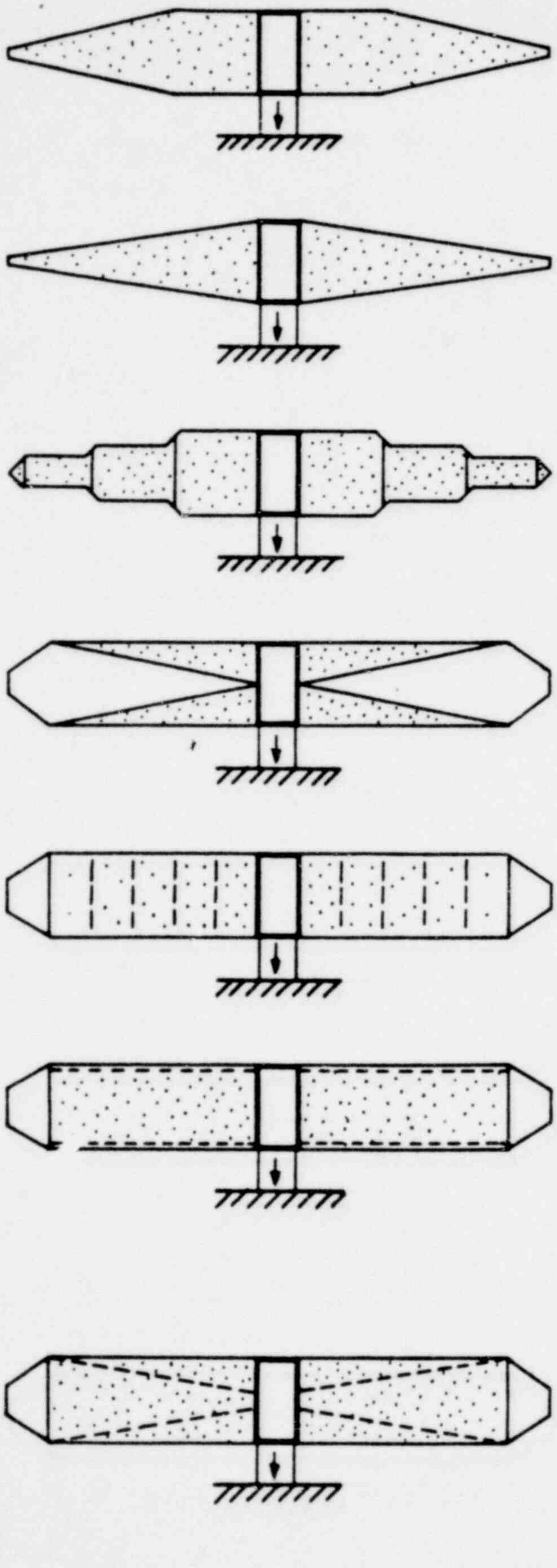

(a) Variable cross-section
Constant open space area

(b) Variable cross-section
Constant open space area

(c) Variable cross-section
Constant open space area

(d) Constant cross-section
Constant open space area
Impermeable internal core

(e) Constant cross-section
Variable open space area

(f) Outer screen:
Constant cross-section
Constant open space area
Internal screen:
Constant cross-section
Variable open space area

(g) Outer screen:
Constant cross-section
Constant open space area
Internal screen:
Variable cross-section
Constant open space area

Figure 5: Various alternative designs of perforated-pipe intake unit.

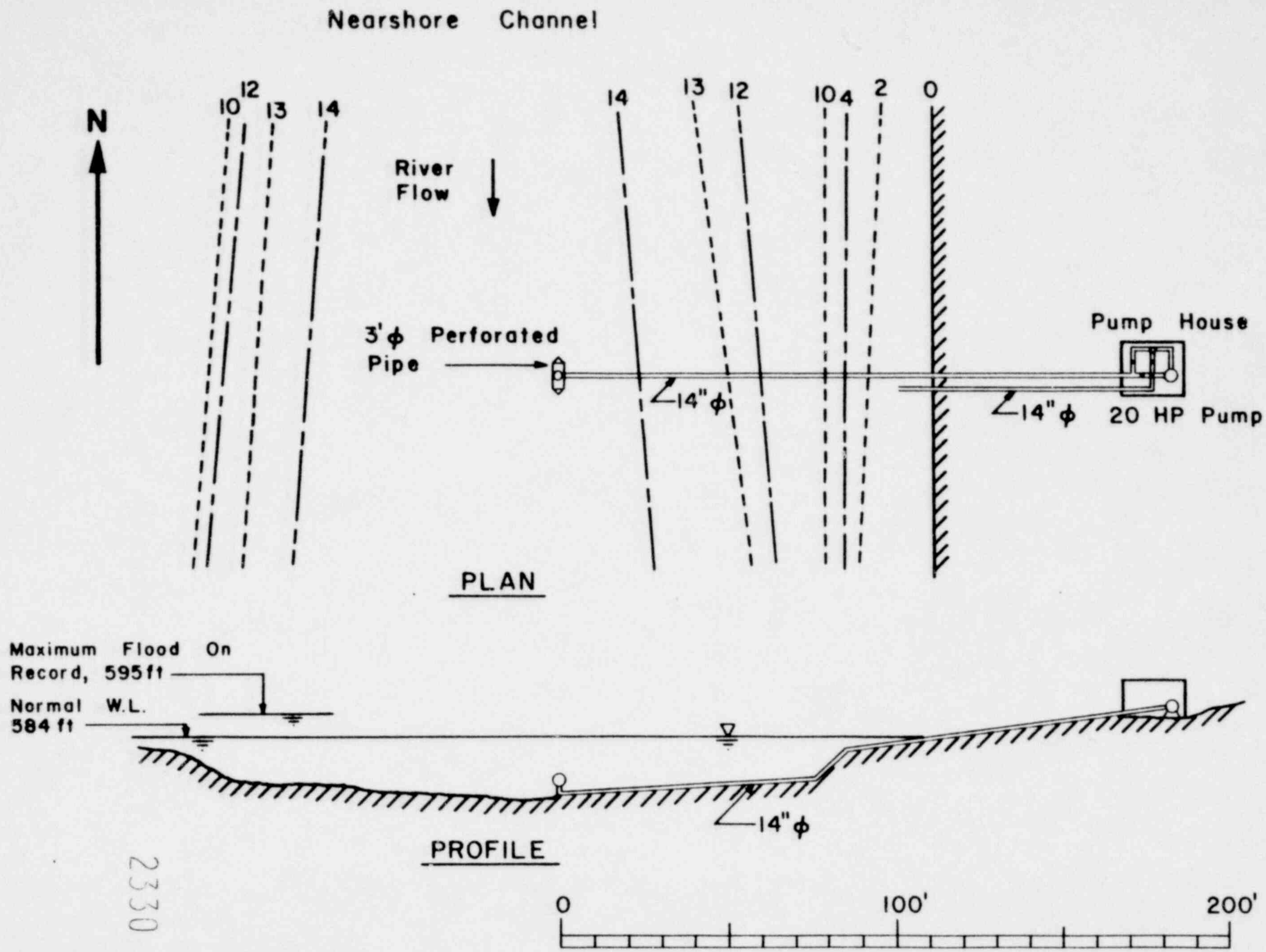

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Figure 6. Layout of field test; plan and profile

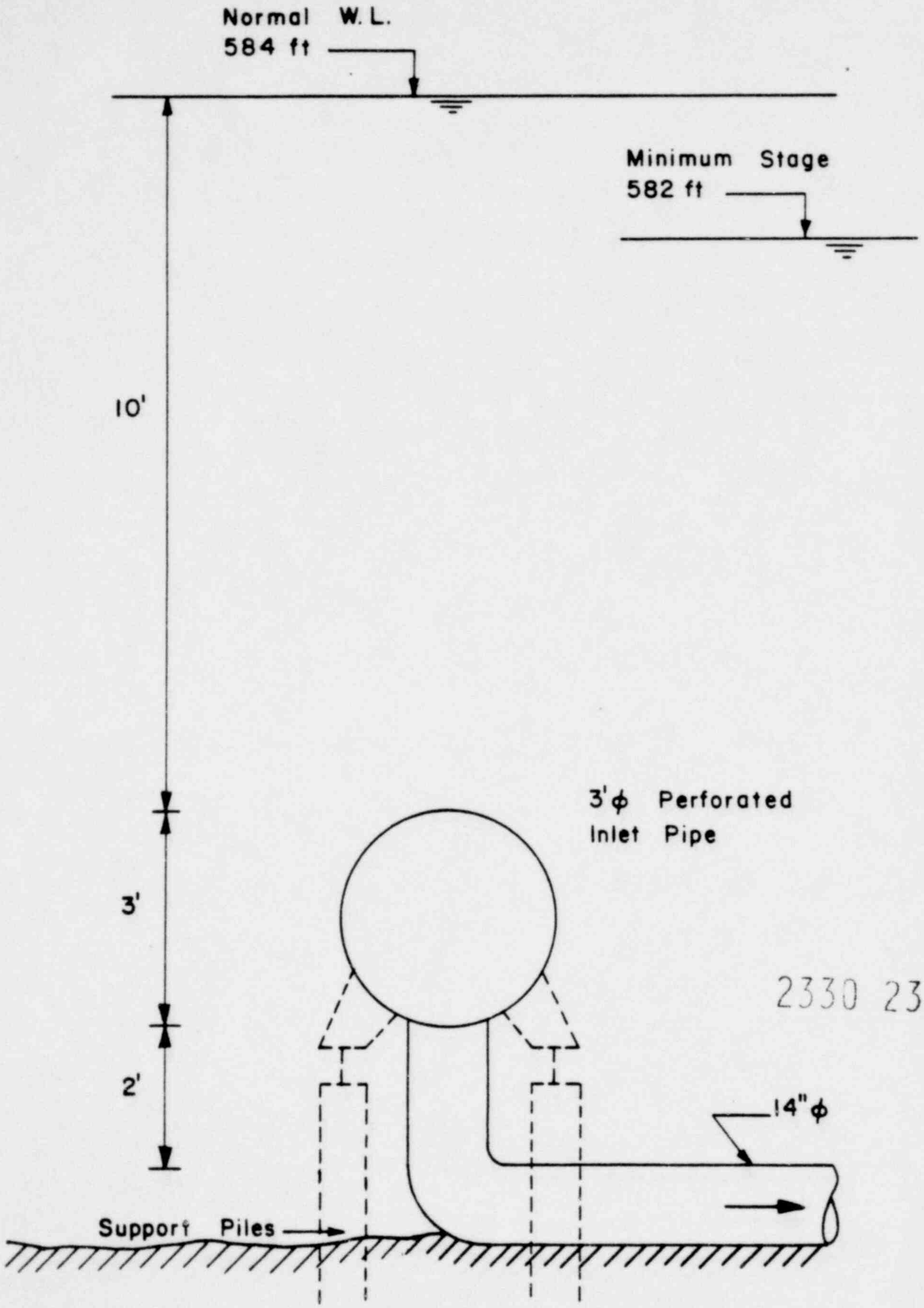

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Figure 7. Layout of field test; cross-section at the perforated pipe inlet

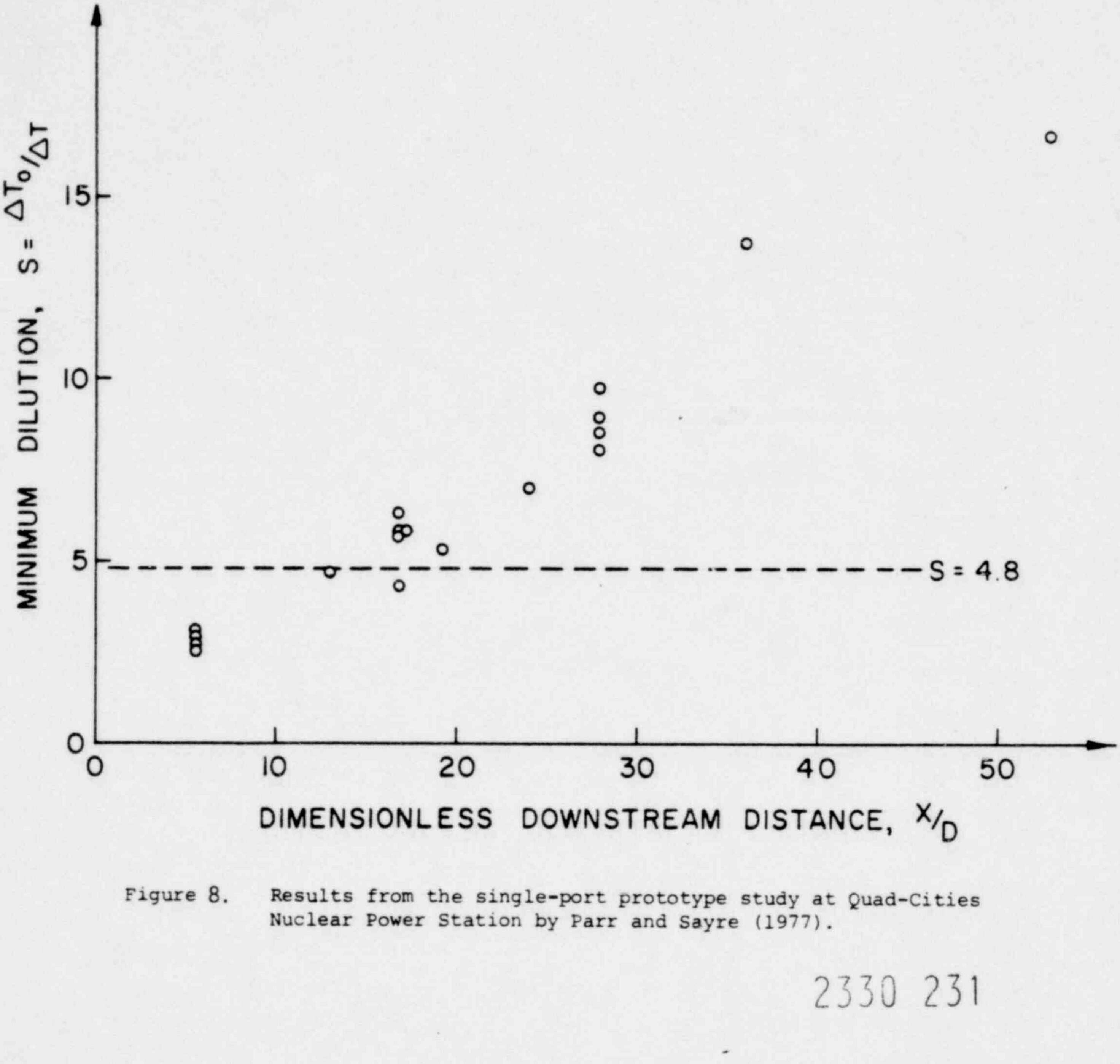

Figure 8. Results from the single-port prototype study at Quad-Cities Nuclear Power Station by Parr and Sayre (1977).

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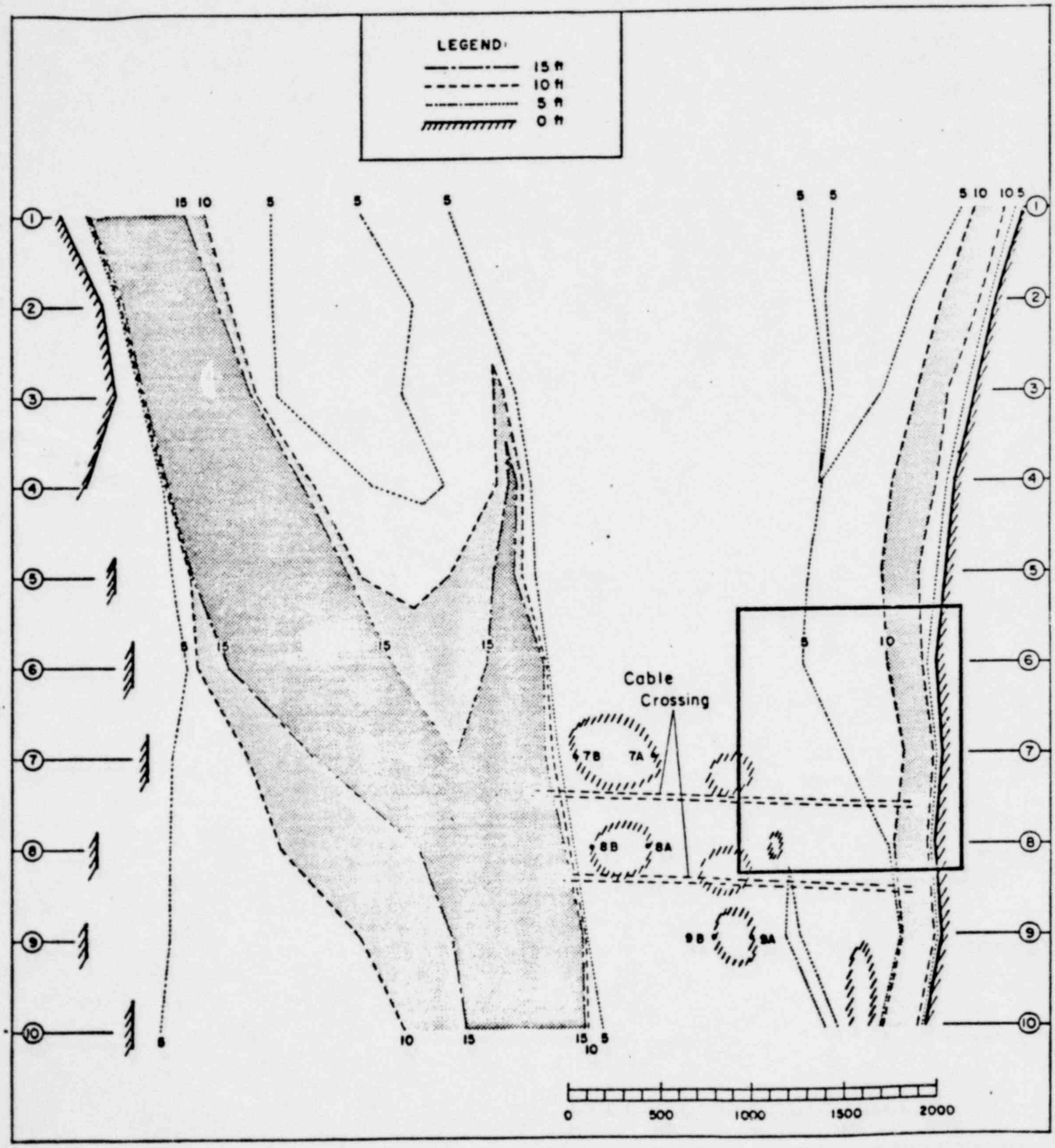

Figure 9: Topographical map showing the area to be modeled.

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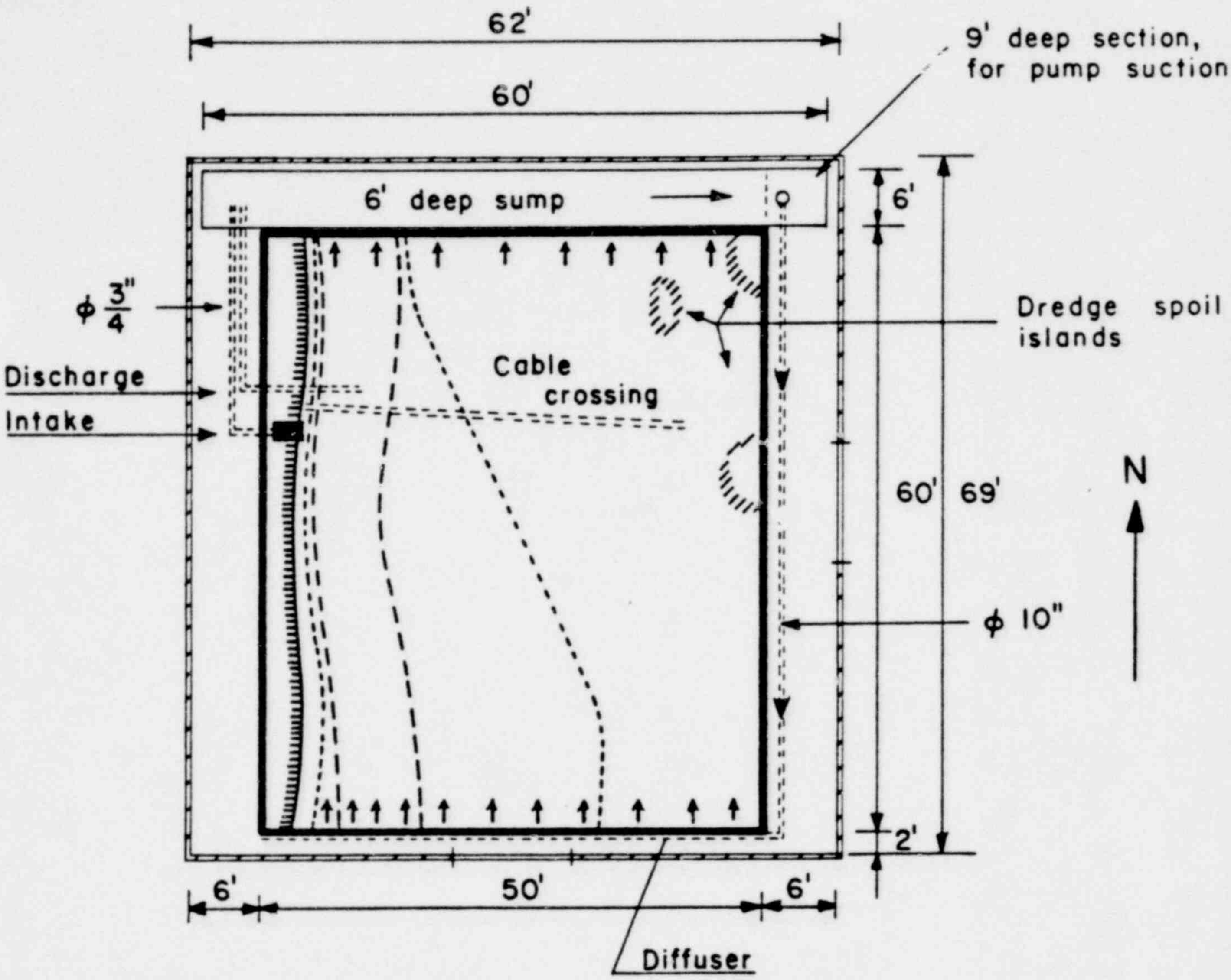

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Figure 10: Model layout

PART THREE

PHASE III. LABORATORY MODEL INVESTIGATION OF THE FLOW PATTERN AROUND PROPOSED INTAKE AND DISCHARGE STRUCTURES

2330 234

May 1979

ABSTRACT

A 1300 ft long and 1200 ft wide area of the Mississippi River has been modeled with the purpose of investigating the effect of proposed intake and discharge structures on the flow pattern in the river. The model scale was 1:24. Two river flow conditions were modeled: The 7-day 10-year low flow for April and May at normal pool elevation (583 ft above MSL), and the annually based 7-day 10-year low flow at low water elevation (582 ft above MSL). The model investigation provided information on the flow pattern around two alternative intake structures.

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COOLING-WATER-INTAKE AND BLOWDOWN-DISCHARGE STUDY
FOR CARROLL COUNTY POWER STATION, PHASE III

I. INTRODUCTION

This report presents the results of Phase III of the cooling-water-intake and blowdown-discharge study for Carroll County Power Station. The investigation was commissioned by Commonwealth Edison Company, Chicago, Illinois, and has been carried out in accordance with the proposal submitted to CECo by IIHR 10 October 1977.

The purpose of the study was to establish a basis for the design of intake and discharge structures such that

- the structures will have good hydraulic performance;
- the structures will require minimum maintenance;
- thermal and other environmental standards will be met.

The intake and discharge structures are to be located on the Mississippi River at River Mile 530, slightly north of Thomson, Illinois. The anticipated maximum intake rate is between 70 and 95 cfs. Conservatively, 100 cfs was used for this study. The anticipated maximum blowdown discharge is between 10 and 30 cfs. Conservatively, 60 to 80 cfs was used in the study.

Phase III includes a laboratory model investigation of the flow pattern around proposed intake and discharge structures.

II. PROTOTYPE DESCRIPTION

The area modeled was 1300 ft long and 1200 ft wide, and located as shown in figure 1. The southern boundary of the model area was 450 ft downstream from the proposed location of the intake structure. The eastern boundary was approximately 90 ft east of the shoreline. The western boundary ran just west of the inner dredge spoil islands. The discharge through this area is approximately 8% of the total river discharge (refer to Phase I report).

Two intake structures were modeled. Both were conventional shoreline intakes of a configuration as proposed by Sargent and Lundy (SL), Chicago. Intake structure no. 1 was modeled after SL's drawing no. M-16, Rev. 2 of October 4, 1978. Its northern (upstream) wall was 115 ft south of transect no. 7 (refer to Phase I report) at coordinate N1,943,237 ft (SL's drawing no. CS-68, Rev. 6 of October 26, 1978). The width of intake structure no. 1 was 64 ft. The net-width at the proposed position of the traveling screens was 45.5 ft. Intake structure no. 2 was modeled after SL's drawing no. MS-10. Its northern wall

was 125 ft south of transect no. 7 at coordinate N1,943,227 ft (SL's drawing no. CS-68, Rev. 2, August 3, 1978). The width of intake structure no. 2 was 53 ft with a net-width at the proposed position of the traveling screens of 39.2 ft.

The discharge structure modeled was a 36-in. diameter pipeline extending 150 ft into the river to a depth of 15 ft below normal pool elevation (583 ft above MSL). This depth is the maximum depth of the 300-ft wide nearshore channel running along the east bank of the river. The top of the discharge nozzle was 10 ft below normal pool elevation (SL's drawing no. CS-74, June 7, 1978). The pipeline was buried below the river bed at a distance of 150 ft south (downstream) of the southern wall of the intake structure (SL's drawing no. CS-68, Rev. 2, August 3, 1978). The alignment of the pipeline was normal to the shoreline (i.e., approximately east-west).

III. MODELING CONSIDERATIONS

To properly simulate the kinematics and dynamics of the flow field, an undistorted model was chosen. Since inertial and gravitational forces are the dominant forces in the flow problem, model and prototype should have the same ratio of inertial forces to gravitational forces. This ratio is expressed by the Froude number

$$F = \frac{V}{\sqrt{gd}}$$

where

V = velocity

d = flow depth

g = acceleration of gravity

In a Froude model, the Froude number of the model,

$$F_m = \frac{V_m}{\sqrt{gd_m}}$$

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is equal to the Froude number of the prototype,

$$F_p = \frac{V_p}{\sqrt{gd_p}}$$

Using $d_m/d_p = L_r$ where L_r is the length scale ratio,

$$V_r = \sqrt{L_r}$$

Subscript r denotes the ratio between model (m) and prototype (p); i.e., $V_r = V_m/V_p$. The discharge ratio, Q_r, for the model to prototype is

$$Q_r = Q_m/Q_p = A_r V_r$$

where

$$A_r = L_r^{\,2}$$

Hence,

$$Q_r = L_r^{\,5/2}$$

To insure a sufficient level of turbulence the Reynolds number,

$$Re_m = \frac{V_m d_m}{\nu}$$

(ν = kinematic viscosity) should be greater than 1500 in the nearshore channel. That means, the length scale ratio, L_r, should be greater than 1:30. A value of 1:24 was found to be reasonable.

The velocity scale ratio is then

$$V_r = 1/\sqrt{24} = 1/4.9$$

and the discharge ratio

$$Q_r = 1/(24)^{5/2} = 1/2822$$

The bottom roughness of the model was determined on the basis of a friction factor ratio, f_r, of 1 between model and prototype; i.e.,

$$f_r = f_m/f_p = 1$$

The friction factor, f, is defined by the relation

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$$V = \sqrt{8g/f}\ \sqrt{RS_f}$$

where R is the hydraulic radius, S_f is the energy slope (the slope of the

energy line), and V is the mean or depth averaged velocity. The friction factor, f, is related to the roughness coefficient, n, in the "Manning" formula

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2}$$

(in English units) by the relation

$$n = 1.49 R^{1/6} \sqrt{f/8g}$$

The value of n recommended by U.S. Army Corps of Engineers (Rock Island District) for the prototype area is 0.028 (±0.002). For the nearshore channel, in which R is roughly 8 ft, this value of n corresponds to a value of f of 0.045 (±0.007). Since f_r=1, the model friction factor, f_m, should also be 0.045 (±0.007). According to the pipe friction factor (Moody) diagram the relative roughness of the model bed should then be approximately 0.01, and the absolute roughness, k, be

$$k = 0.01(4R) \cong 0.013 \text{ ft} \cong 4.0 \text{ mm}.$$

To obtain this roughness well-packed sand #3 was chosen. As seen in figure 2 this sand has a median diameter of 3.7 mm.

IV. THE MODEL

A. Construction. The model was constructed within a 50-ft by 57-ft concrete-block-wall basin at the Institute of Hydraulic Research. A schematic plan view of the model is shown in figure 3.

The topography of the river bed was modeled by filling the spaces between wooden templates with well-packed sand #3. The templates, which were cut to represent the shape of the river bed as measured in surveys in September 1977 and April 1978 (refer to Phase I report), were spaced at 2-ft to 8-ft intervals.

Water was supplied to the model from a 6 ft deep, 6 ft wide, and 60 ft long storage sump at the downstream end of the model. The water was pumped from the sump through a 10-in. diameter pipe to a 50 ft long 10-in. diameter diffuser extending across the upstream end of the model. The diffuser was placed

in a 33-in wide canal separated from the model by hair mats. The water returned to the sump after having passed through the model and over a tail-gate weir at the downstream end of the model.

The tail gate consisted of adjustable segments. It was used to adjust the discharge distribution across the model (by adjusting the elevation of the individual segments) and to control the depth of flow in the model. Adjustments of the discharge distribution could also be made by adjusting the port spacing in the diffuser and the thickness of the hair mats between the model and the diffuser canal.

The discharge was controlled by an automatic flow control system mounted on the pipeline between sump and diffuser. The control system consisted of a butterfly valve, an actuator, and a controller. A spring inside the actuator applied a force that tended to close the valve. The valve opened when air pressure against a diaphragm on top of the spring exceeded the spring force. The air pressure was controlled by the pressure differential at an orifice meter placed 6 ft upstream from the valve.

Figures 4 and 5 show the intake structures. Both structures consisted of three channels, 7.5-in. wide (15-ft wide in the prototype) in intake no. 1, 6.5-in. wide (13-ft wide in the prototype) in intake no. 2, with three 2-in. diameter suction pipes (48-in. diameter suction pipes in the prototype). The water depth in the intakes was 4-in. (8 ft in the prototype) at normal pool elevation. The intake channels were normal to the shoreline. The face of the channels and the proposed position of the traveling screens flushed with the shoreline at normal pool elevation. Four support piles were placed in the river at a distance of 9 in. (5.5 in. for intake no. 2) from the face of the intake channels.

B. Measuring Techniques. The flow rates through the intake and discharge lines were measured by means of calibrated orifice meters and two-tube precision manometers.

The velocities were measured with a miniature propeller current meter and floating wooden cubes.

The current meter is shown in figure 6 (which also includes a view of model intake structure no. 1). The probe used is shown in figure 7. The output from the probe, which is the frequency of revolutions of the propeller (the number of revolutions in a fixed time interval of 1, 10, or 100 sec), is displayed by an electronic, digital counter.

The current meter was calibrated in a 3-ft wide and 30-ft long towing tank. The probe was mounted on a carriage which would be driven by a variable speed motor on rails fastened to the walls of the tank. The calibration curve was obtained by moving the carriage (and the propeller) at given velocities and recording the corresponding frequencies of the propeller revolutions.

In certain areas of the model the velocities were too low to be determined by the propeller current meter, especially in low flow situations. In such cases the velocities had to be determined via the float velocity The depth-average velocity, V, was then determined by multiplying the float velocity, U_s, by the factor 0.84.

The float velocities were determined either by measuring the time (by stop watch) required for a small ¼-in. by ¼-in. wooden cube to travel a given distance or by measuring the distance traveled by the cube in a fixed time interval. The latter was done photographically by placing a camera 10 ft above the water surface and exposing the film over periods of 10 sec. In this case the measurements were facilitated by a 6-in. by 6-in. mesh coordinate net placed 0.5 in. above the water surface.

The factor 0.84 was estimated on the basis of the Prandtl universal logarithmic velocity-distribution law for pipes:

$$U = 2.5\, U_f \ln y + c$$

where U is the velocity at a distance y from the pipe wall, and U_f is the friction velocity given by

$$U_f = \sqrt{\tau_o/\rho} = V\sqrt{f/8} \tag{1}$$

τ_o is the shear stress at the pipe wall, and ρ is the density. Eq. (1) is also a good approximation for the velocity profile in a two-dimensional open channel. Since $U = U_s$ for $y = d$, Eq. (1) may be written

$$U - U_s = 2.5\, U_f \ln(y/d) \tag{2}$$

The depth-average velocity, V, can be obtained by integrating this equation over the depth:

$$V - U_s = -2.5\, U_f \tag{3}$$

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Note from Eqs. (2) and (3) that $U = V$ when $y = 0.368d \approx 0.4d$. Substituting $V\sqrt{f/8}$ for U_f in Eq. (3) the result is

$$\frac{V}{U_s} = \frac{1}{1 + 0.88\sqrt{f}}$$

or, since $f \approx 0.045$,

$$\frac{V}{U_s} = 0.84 \qquad (4)$$

This result was checked during the model calibration. The velocities in the calibration flow were high enough to allow the velocities in the nearshore channel to be measured by the propeller current meter. Velocities were measured at three different levels below the surface of the water: at $y = 0.2d$, at $y = 0.4d$, and at $y = 0.8d$. The depth-average velocity, V, was determined by taking the average of (1) the velocity, $U_{0.4}$, at $y = 0.4d$, and (2) the average of the velocities at $y = 0.2d$ and $y = 0.8d$; i.e.,

$$V = 0.5\, U_{0.4} + 0.25(U_{0.2} + U_{0.8}) \qquad (5)$$

The results are listed in table 1 and compared with the float velocities, U_s, as determined by the floating wooden cubes and a stop watch. The results compare favorably with the theoretical result above, Eq. (4).

The directions of the velocity vectors were determined either directly from the photographs of the streak lines traced out by the floating wooden cubes or by lowering a piece of thread down in the water column to the level of measurement and measuring the angle between the thread and the strings of the coordinate net above the water surface.

C. Calibration. The model was calibrated against the results of the field measurements along transect no. 7 (refer to Phase I report). Transect no. 7 is located 115 ft north (upstream) of the proposed position of intake no. 1 and 624 ft north of the southern boundary of the model area. It was profiled on September 19, 1977, and velocities along the section were measured on October 12, 1977. The stage at the intake was determined on the basis of the stages at Sabula and Clinton (by linear interpolation). On both days the stage at the intake was 583.32 ft above MSL. On the day of the velocity measurements, the river discharge was 44,400 cfs. of which 8 %

or 3550 cfs passed through the model area. The results of the prototype velocity measurements are shown in figure 8.

The calibration discharge in the model was 1.33 cfs, corresponding to a prototype discharge of 3200 cfs. The flow depth in the model corresponded to a river stage of 583.00 ft at the intake, The velocities along transect no. 7, measured in the final calibration test, are shown in figure 9.

Figure 10 shows the distributions of the cumulative percentage of flow through transect no. 7 in the model and the prototype. The distributions appear quite similar demonstrating that the model is able to reproduce the prototype velocity distribution across the transect no. 7.

All results from the final calibration test are listed in table 2. The velocities listed are the float velocities measured in the model multiplied by the velocity scale ratio 4.9. The directions of the velocity vectors are shown in figure 11. The calibration section is located 624 ft upstream from the southern boundary. At this section the shoreline is 90 ft from the eastern boundary.

The vertical velocity profiles in the model also were compared with the profiles in the prototype along transect no. 7. In the prototype the ratio $U_{0.8}/U_{0.2}$ between the velocity at y = 0.8d and the velocity at y = 0.2d varied between 1.05 and 1.46 around an average of 1.26, while those in the model varied between 1.06 and 1.29 around an average of 1.18. The agreement is satisfactory and shows that the velocity profiles in the prototype and the model are reasonably similar to each other.

V. Results

The purpose of the modeling was to investigate the changes in the flow pattern in the river around the proposed intake and discharge structures. Two river flow conditions were modeled: The 7-day 10-year low flow for April and May at normal pool elevation (583 ft above MSL), and the annually based 7-day 10-year low flow at low water elevation (582 ft above MSL).

As seen in figure 12 and 13 the elevation 583 ft is the 9th percentile of the river stage at the proposed position of the intake based on daily means for April and May over the period 1947 to 1977, while the elevation 582 ft is the 5th percentile of the stage at the intake based on daily means for the

whole year over the period 1947 to 1977. Figure 14 and 15 show the correlation between discharge and stage at the intake. Marks indicate the flow situations modeled. Figure 14 and 15 are prepared by plotting corresponding values of stage and discharge for each day of the year (figure 15) or each day of the months April and May (figure 14) over the years 1947 through 1977.

Initially it was checked if the excavation around the intake structures would change the present flow pattern significantly. The proposed structures would involve the excavation of up to 15,000 ft^3 of sediment material from the river bed in front of the intake increasing the water depth to 8 ft at normal pool elevation. The checking was done by observing the behavior of floating wooded cubes before and after the excavation. It was found that the excavation did not change the present flow pattern significantly. The body of water at the proposed site of the excavation is almost stagnant, and the only observable effect on the flow pattern of the excavation was a slightly increased tendency to eddy formation in the area. Occasionally the eddy formation resulted in a slow return current along the shoreline in the upstream direction.

A. The 7-Day 10-Year Low Flow for April and May. Intake No. 1.

At the 7-day 10-year low flow for April and May the discharge through the model area is 2200 cfs with 1300 cfs passing through the 300-ft wide nearshore channel.

The area affected by the withdrawal of water through the intake was determined on the basis of profile measurements at 16 points across the nearshore channel at the position of the northern wall of the intake. At each point the velocity was measured at three different levels below the surface of the water, at y = 0.2d, y = 0.4d, and y = 0.8d. The depth averaged velocity was determined by Eq. (5). The results are listed in table 3 and shown in figure 16. It was found that the depth averaged velocity at the shoreline increased from less than 0.1 fps at no withdrawal to approximately 0.4 fps in case of withdrawal. At a distance of 47 ft from the shoreline (or 24 ft from the outer support pile) the velocity increased from 0.29 fps at no withdrawal to 0.38 fps in case of withdrawal. Figure 16 also shows that the maximum depth-average velocity in the nearshore channel at the position of the intake was measured to be 0.50 fps.

Figure 17 shows the distributions across the nearshore channel of the discharge per unit width with and without withdrawal. The distribution

is along the same section as the velocity distribution in figure 16. Figure 17 shows that the area affected by the withdrawal extended outward to a distance of approximately 80 ft from the shoreline (or approximately 60 ft from the outer stop log piles of the intake). By measuring the area below the curves it is found that the discharge entering the affected area increased from 160 cfs to 260 cfs as a result of the withdrawal.

A more detailed investigation of the flow pattern just outside the outer support piles was carried out in connection with the modeling of intake structure no. 2. The results are presented in Section C of Chapter V.

The velocity distribution in the immediate vicinity of the intake structure (within a distance of 20 ft from the proposed position of the traveling screens) was determined on the basis of profile measurements at the points shown in figure 18. The points are either center or quarter points. For example, point no. 5 is halfway between the walls of the intake channel; point no. 4 is halfway between the wall and point no. 5. At each point the velocity was measured at three different levels below the surface of the water. The results are listed in tables 4 and 5. Table 4 shows the velocities measured when the make-up water (100 cfs) was withdrawn through intake channels no. 1 and 2 (pumps 1 and 2 in operation). Table 5 shows the velocities measured when the water was withdrawn through intake channels no. 2 and 3 (pumps 2 and 3 in operation). The tables are arranged such that each velocity profile appears at the approximate position of the point of measurement relative to the intake structure. The position of the stop log piles and channel walls are indicated in the tables. Below the number indicating the point of measurement, four velocities are listed: (1) the velocity at a depth below the surface of the water of 0.2 times the depth of water; (2) the velocity at a depth of 0.5 times the depth of water; (3) the velocity at a depth of 0.8 times the depth of water; and (4) the average value of (1), (2), and (3). The arrow after each velocity indicates the direction of the velocity vector. An arrow pointing downward indicates that the velocity is directed toward the shoreline (or the intake channel); an arrow pointing to the left indicates that the velocity is directed in the downstream direction parallel to the shoreline and the face of the intake. Figures 19 and 20 show the corresponding depth averaged flow pattern. Table 6 gives an account of the discharge distribution around the intake structure in the two cases considered.

The tables 4 and 5 show that the velocity distribution across the intake channels (at the proposed position of the traveling screens) was significantly nonuniform. In the case of withdrawal through the channels 1 and 2 (table 4) the velocity variation was ± 15% across channel no. 1 while it was ± 25% across channel no. 2. In the case of withdrawal through the channels 2 and 3 (table 5) the velocity variation was ± 29% across channel no. 2 while it was ± 18% across channel no. 3.

Extensive eddy formations were noticed in the area between the support piles and the intake channels. These eddy formations were undoubtedly induced by the flow separation around the support piles. As seen in table 4 and 5 no net velocity could be recorded at the points 21 and 18. Apparently these points were within the wake of the support piles. The eddy formations were undoubtedly a contributing factor to the nonuniformity of the velocity distributions across the intake channels.

B. The Annually Based 7-Day 10-Year Low Flow. Intake No. 1. At the annually based 7-day 10-year low flow the discharge through the model area is 1150 cfs with 680 cfs passing through the nearshore channel.

In this case the area affected by the withdrawal was determined on the basis of float measurements. The velocities were too low to be determined by the propeller current meter. The results are listed in table 7 and shown in figure 21. It was found that the depth averaged velocity (determined as 0.84 times the float velocity) at the shoreline increased from less than 0.1 fps at no withdrawal to approxmiately 0.4 fps in case of withdrawal. At a distance of 47 ft from the shoreline (or 24 ft from the outer support pile) the velocity increased from 0.24 fps at no withdrawal to 0.31 fps in case of withdrawal. Figure 21 also shows that the maximum depth-average velocity in the nearshore channel at the position of the intake was 0.29 fps at no withdrawal.

Figure 22 shows the distribution across the nearshore channel of the discharge per unit width with and without withdrawal. It is seen that the area affected by the withdrawal extended outward to a distance of approximately 120 ft from the shoreline (or approximately 100 ft from the outer support piles of the intake). By measuring the area below the curves it is found that the discharge entering the affected area increased from 180 cfs to 280 cfs as a result of the withdrawal.

The eddy formations in the area between the support piles and the intake channels were not as pronounced as in the 7-day 10-year low flow for April and May. However, the nonuniformity of the velocity distribution increased across the channels 1 and 3. The velocity variation increased from

± 15% to ± 25% across channel no. 1 (at pumps 1 and 2 in operation), and from ± 18% to ± 23% across channel no. 3 (at pumps 2 and 3 in operation) The results are presented in tables 8, 9, and 10 and shown in figures 23 and 24.

C. The 7-Day 10-Year Low Flow for April and May. Intake No. 2. The configuration of intake structure no. 2 is similar to that of intake structure no. 1. However, the width of the intake channels is only 13 ft (15.1 ft for intake no. 1), the distance from the intake channels to the outer stop log piles is 11 ft (18 ft for intake no. 1), and the width of the outer stop log piles is 3 ft (5 ft for intake no. 1).

The velocity pattern around intake no. 2 was determined on the basis of profile measurements at the points shown in figure 25. Outside the outer stop log piles the profile measurements were supplemented by float measurements. The results are listed in the tables 11, 12, and 13, and shown in figures 26, 27, and 28.

Figure 26 shows the velocity pattern in front of the intake with and without withdrawal. At a distance of 25 ft from the intake (from the position of the outer stop log piles) the depth-average deflection of the velocity vector due to the withdrawal was measured to be approximately 10 degrees. At a distance of 50 ft from the intake the deflection was 5 degrees or less.

Figures 27 and 28 show the flow pattern in the immediate vicinity of the intake structure (within a distance of 30 ft from the proposed position of the traveling screens). The velocities shown are depth-average velocities. In figure 27 the make-up water was withdrawan through intake channels no. 1 and 2. In this case the velocity distribution across the entrance to intake channel no. 1 was practically uniform. Across intake channel no. 2 the depth-average velocity varied from 0.43 fps at the upstream quarter point of the section to 0.58 fps at the downstream quarter point. In front of intake channel no. 3 a weak current was noticed in the upstream direction.

Figure 28 shows the near-field flow pattern measured when the make-up water was withdrawn through intake channels no. 2 and 3. In this case the nonuniformity across the downstream intake channel (no. 3) was less pronounced than the nonuniformity across the downstream channel (no. 2) in the case above. Again, the flow across the upstream intake channel (no. 2) was practically uniform.

All results of the near-field float- and profile-measurements are presented in the tables 11, 12, and 13. As will be seen from table 13, the

case of simultaneous withdrawal through intake channels no. 1 and 3 was also investigated. In this case significant (though not severe) nonuniformity was measured across both intake channels. Table 13 also shows that the nonuniformity became less pronounced when the "inactive" channel (no. 2) was sealed off by stop logs according to the design.

D. The Annually Based 7-Day 10-Year Low Flow. Intake No. 2. Results from this flow situation are presented in tables 14 and 15 and shown in figure 29. The flow pattern in this case was very similar to that of the 7-day 10-year low flow for April and May. Apart from a general increase of the velocities due to the decrease in water depth and a slight change of the direction of the velocity vectors, the only difference was a slightly higher degree of nonuniformity of the velocity distribution across the upstream intake channel (channel no. 2 when pumps 2 and 3 were in operation, and channel no. 1 when pumps 1 and 2 were in operation). Across the intake channels the velocity varied from a minimum of 0.48 fps to a maximum of 0.72 fps with an average of 0.57 fps.

E. The Blow-Down Discharge. The effect of the blow-down discharge on the flow pattern was determined by profile measurements and by injecting dye solutions into the discharge pipe and observing the area being colored. The investigation was made for a discharge of 80 cfs, and for three different orientations of the discharge nozzle. When the nozzle was pointing (1) upstream, (2) downstream, and (3) away from the shoreline toward the navigation channel.

When the nozzle pointed upstream the affected (colored) area extended approximately 175 ft upstream from the nozzle at the annually based 7-day 10-year low flow. In this case colored fluid was drawn into the intake. At the 7-day 10-year low flow for April and May the colored area extended approximately 125 ft upstream from the nozzle. When the nozzle pointed away from the shoreline toward the navigation channel the colored area never extended more than 30 ft upstream from the discharge nozzle. When the nozzle pointed downstream the jet effect caused a slight increase of the velocity right upstream from the nozzle; see table 16. However, as will be seen in table 16 there was no significant increase in the velocity at a distance of 75 ft upstream from the nozzle. Downstream from the nozzle the effect of the discharge is, of course, very significant. A discussion of this case was

given in the Phase II report. No attempt was made to model this situation since the area of influence would extend far beyond the southern boundary of the model.

VI. CONCLUSION

The model investigation has shown that the effect of the proposed intake and discharge structures on the flow pattern in the river is very limited. At the annually based 7-day 10-year low flow the discharge through the 300-ft wide nearshore channel is approximately 680 cfs of which up to 100 cfs will be withdrawn through the intake. The investigation showed that the area affected by the withdrawal in this case would extend outward to a distance of approximately 120 ft from the shoreline.

The velocity at the shoreline can be expected to increase from less than 0.1 fps to approximately 0.4 fps due to the withdrawal, which may give rise to scouring of the river bed in the vicinity of the intake.

The investigation included measurements of the velocity pattern around two alternative intake structures (no. 1 and no. 2). Both structures were shoreline structures. They consisted of three intake channels normal to the shoreline. The face of the intake channels flushed with the shoreline at normal pool elevation. Four support (stop log) piles were placed in the river at a certain distance from the intake channels.

At intake structure no. 1 the flow separation around the support piles caused extensive eddy formations in the area between the piles and the intake channels. The velocity distribution across the intake channels was significantly nonuniform. In one case a velocity variation of ± 29% was observed. The nonuniformity was undoubtedly, to some extent, due to the presence of the support piles.

The eddy formations were less pronounced at intake structure no. 2, and so was the nonuniformity of the velocity distributions across the intake channels. Intake no. 2 was smaller tha[illegible]e no. 1. The width of the intake channels was only 13 ft (15.1 ft for intake no. 1), the distance from the intake channels to the support piles was 11 ft (18 ft for intake no. 1), and the width of the support piles was 3 ft (5 ft for intake no. 1).

The results indicate that an optimization of the hydraulic performance of the intake structure might be achieved by minimizing the horizontal

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dimensions of any support pile that obstructs the river flow and possibly by changing the orientation of the intake structure. To properly simulate the flow pattern within the intake structure for different orientations of the structure it is, however, recommended that a separate model study be carried out at a larger scale, for example 1:8 or 1:10.

Both intake structures are expected to have acceptable hydraulic performance. Due to the relatively low velocities in the flow field around the structures, the effect of the eddy formations on the performance is believed to be minor

At intake structure no. 1 the non-uniformity of the velocity across the screen face decreased when the pumps at Channel 1 and Channel 2 were used for withdrawal rather than the other combinations. This is illustrated in Table 17. To minimize the impact on aquatic life, this is recommended to be the mode of normal operation for intake structure no. 1. At intake structure no. 2 it made no significant difference in the degree of non-uniformity whether the combination was 1 and 2 or 2 and 3. However, the non-uniformity increased when the combination was 1 and 3.

At intake structure no. 1 the approach velocity at the screen face averaged a value of 0.5 fps at the annually based 7-day 10-year low flow. This value was exceeded only locally. At intake structure no. 2 the approach velocity at the screen face averaged a value of 0.57 fps at the annually based 7-day 10-year low flow.

Intake structure no. 1 is believed to be designed such that it will require minimum maintenance.

Distance from eastern boundary (ft)	Distance from shoreline (ft)	Velocity (fps) at level:			Depth-averaged velocity, V	Float velocity, U_s (fps)	V/U_s
		0.8d	0.4d	0.2d			
144	54	.63	.58	.49	.58	.69	.84
168	78	.76	.69	.63	.69	.88	.78
192	102	.82	.74	.65	.73	.93	.78
216	126	.73	.70	.65	.70	.86	.81
240	150	.73	.65	.60	.66	.81	.82
288	198	.67	.62	.57	.62	.69	.90
336	246	.57	.55	.53	.55	.71	.77
384	294	.58	.54	.52	.55	.65	.85
432	342	.62	.58	.55	.58	.62	.94

Table 1. Velocity measurements along transect no. 7 in the calibration flow; check of the ratio between the depth averaged velocity and the float velocity

Distance from eastern boundary (ft)	Distance from southern boundary (ft)						
	240	336	432	528	624	864	1104
104			0.25				
120		0.36	0.42	0.28	0.25		
144	0.51	0.64	0.71	0.68	0.69	0.35	0.45
168	0.77	0.78	0.86	0.92	0.88	0.88	1.14
192	0.88	0.87	0.91	0.99	0.93	1.08	1.12
216	0.84	0.85	0.83	0.88	0.86	1.00	1.01
240	0.77	0.77	0.78	0.83	0.81	0.98	1.19
288	0.69	0.69	0.74	0.79	0.69	0.75	0.66
336	0.59	0.60	0.70	0.73	0.71	0.68	0.68
384	0.49	0.53	0.64	0.72	0.65	0.70	0.70
432	0.47				0.62		0.69
552	0.49				0.61		0.23
720	0.49				0.47		<0.1
936	0.46				0.41		0.35
1080	<0.1				0.36		0.37
1152	0.58						0.37

Table 2. Velocity measurements (float velocities) in the calibration flow

Distance from shoreline (ft)	Distance from intake (ft)	Depth of water, d (ft)	Velocity (fps) at level: y=0.8d	y=0.4d	y=0.2d	Depth-average velocity (fps)	
35	12	8.7	.27	.22	.18	.22	
			.43	.42	.36	.41	*
47	24	10.5	.39	.28	.22	.29	
			.42	.39	.31	.38	*
59	36	10.7	.44	.37	.31	.37	
			.43	.40	.39	.41	*
71	48	10.7	.50	.42	.37	.43	
			.49	.45	.40	.45	*
83	60	10.7	.54	.46	.42	.47	
			.56	.51	.43	.50	*
95	72	11.5	.56	.49	.42	.49	
			.56	.51	.42	.50	*
119	96	12.7	.52	.49	.42	.48	
143	120	14.0	.52	.46	.38	.46	
167	144	14.0	.47	.44	.37	.43	
191	168	13.3	.46	.43	.37	.42	
215	192	9.2	.45	.43	.40	.43	
239	216	6.3	.43	.41	.32	.39	
263	240	6.0	.42	.38	.31	.37	
287	264	6.0	.39	.36	.30	.35	
311	288	6.0	.39	.33	.27	.33	
335	312	5.7	.35	.31	.27	.31	

* with withdrawal

Table 3. Velocity measurements across the nearshore channel at the intake (no. 1) at the 7-day 10-year low flow for April and May; stage = 583 ft

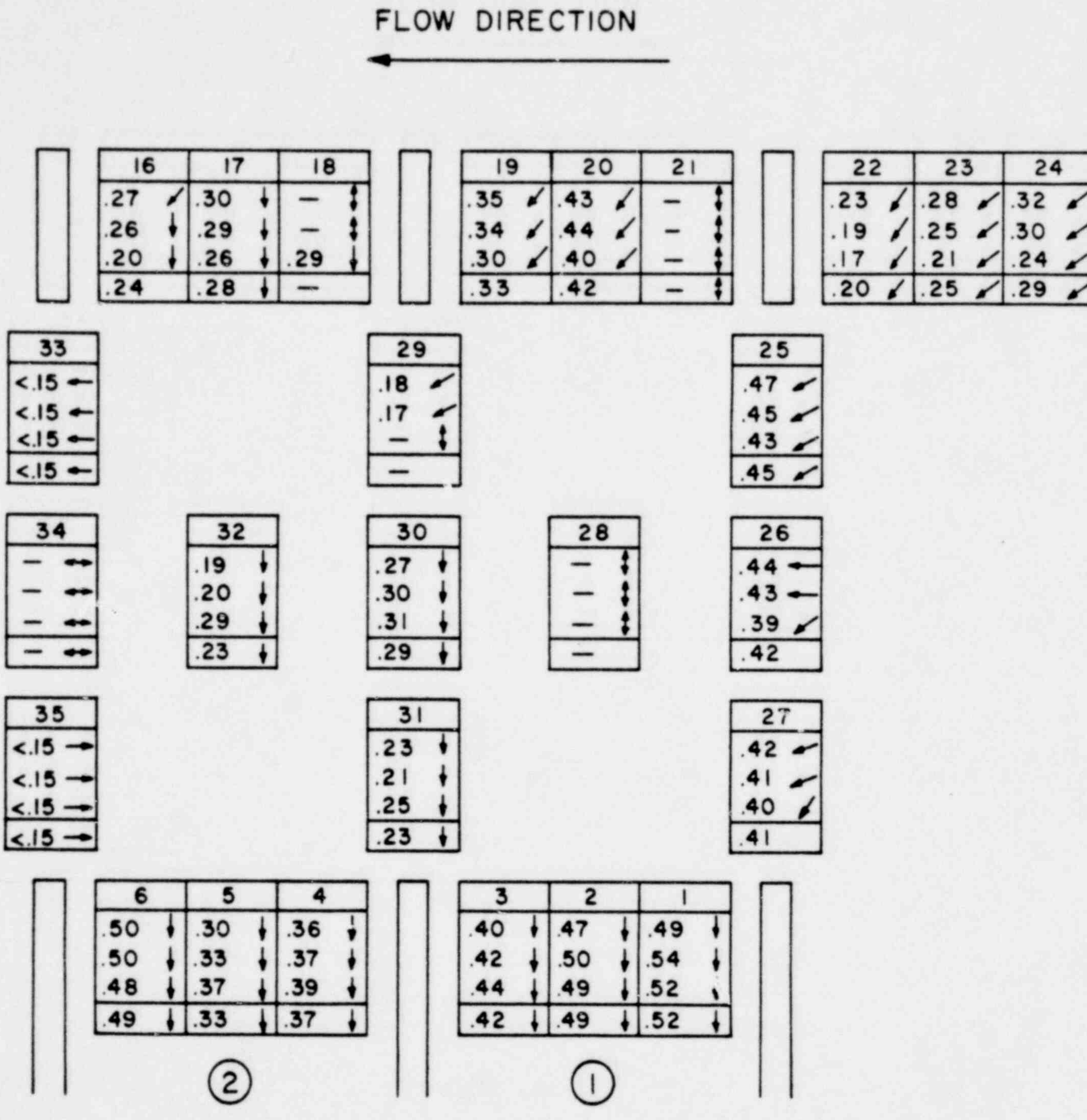

Table 4. Velocity measurements (in fps) at intake structure no. 1 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 1 and 2 in operation

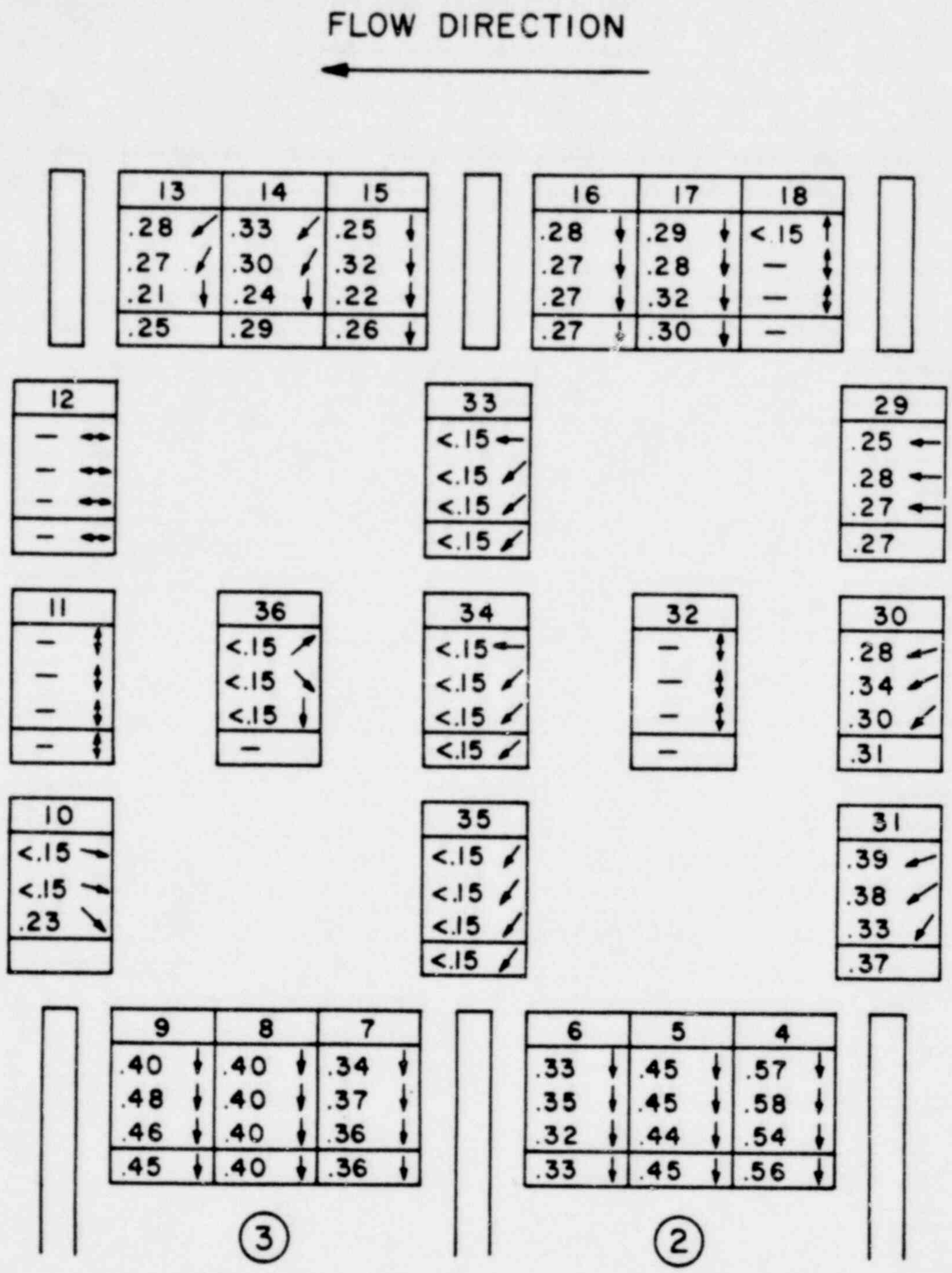

Table 5. Velocity measurements (in fps) around intake structure no. 1 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 2 and 3 in operation

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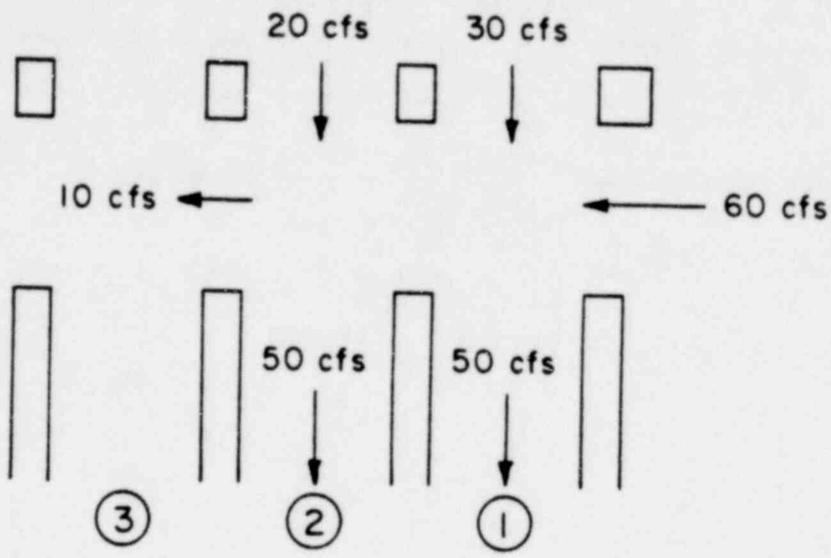

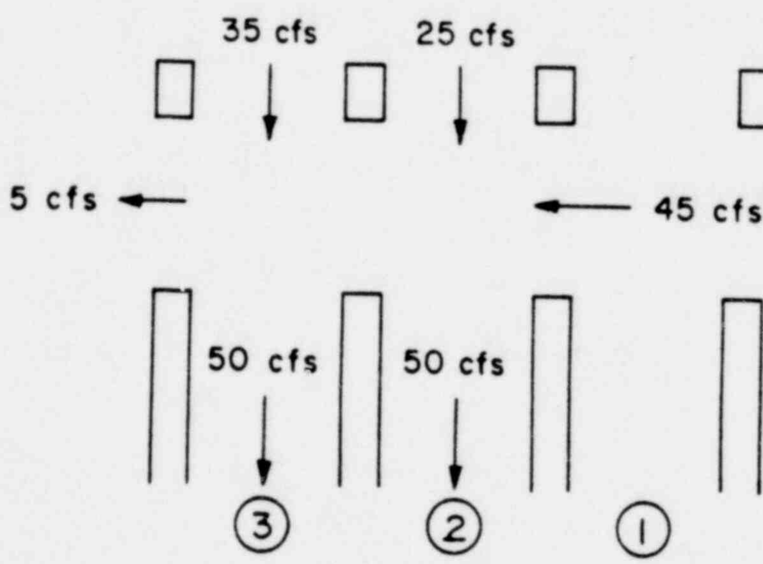

Table 6. Discharge distribution around intake structure no. 1 at the 7-day 10-year low flow for April and May; stage = 583 ft.

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Distance from shoreline (ft)	Distance from intake (ft)	Depth of water, d (ft)	Float velocity U_s (fps)	Depth-average velocity = 0.84 times U_s (fps)
35	12	7.7	.29	.25
47	24	9.5	.28	.24
			.36	.31 *
59	36	9.7	.31	.26
			.32	.27 *
71	48	9.7	.34	.29
			.35	.30 *
83	60	9.7	.33	.28
			.38	.32 *
95	72	10.5	.34	.29
			.39	.33 *
119	96	11.7	.34	.29
143	120	13.0	.33	.28
167	144	13.0	.31	.26
191	168	12.3	.32	.27
215	192	8.2	.32	.27
239	216	5.3	.33	.28
263	240	5.0	.36	.31
287	264	5.0	.34	.29
311	288	5.0	.32	.27
335	312	4.7	.30	.26

* with withdrawal

Table 7. Velocity measurements across the nearshore channel at the intake (no. 1) at the annually based 7-day 10-year low flow; stage = 582 ft

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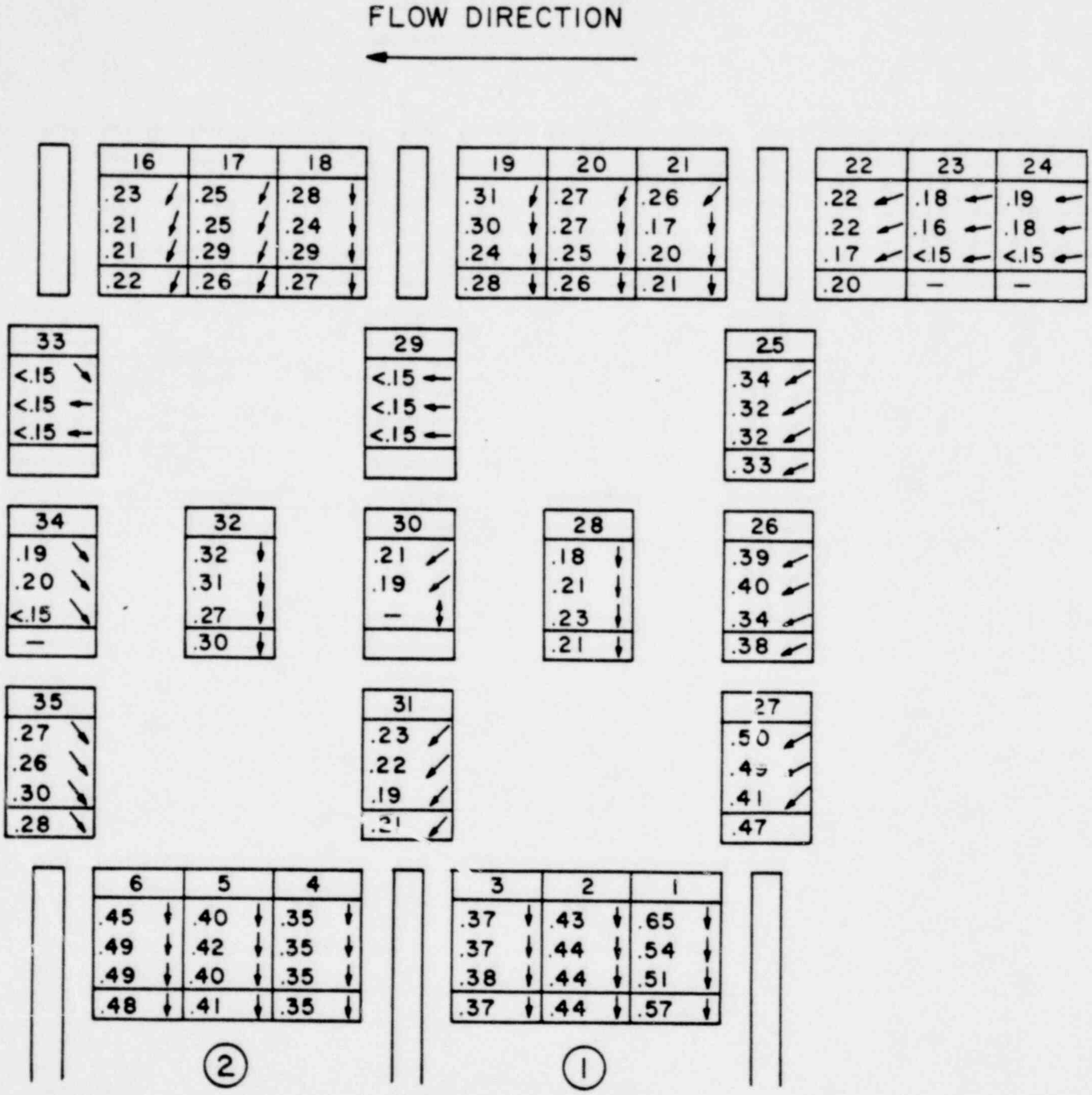

Table 8. Velocity measurements (in fps) at intake structure no. 1 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 1 and 2 in operation

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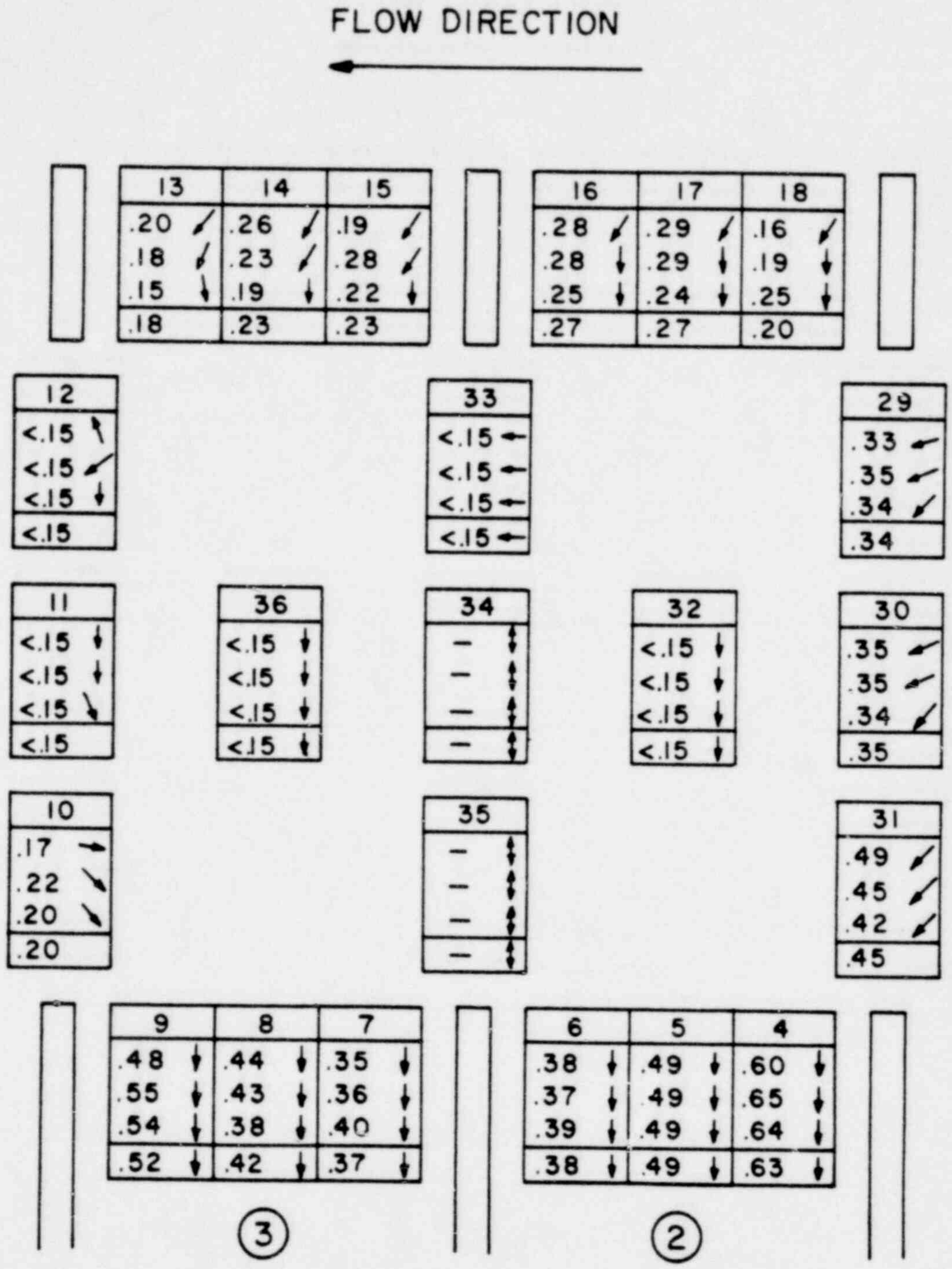

Table 9. Velocity measurements (in fps) at intake structure no. 1 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 2 and 3 in operation.

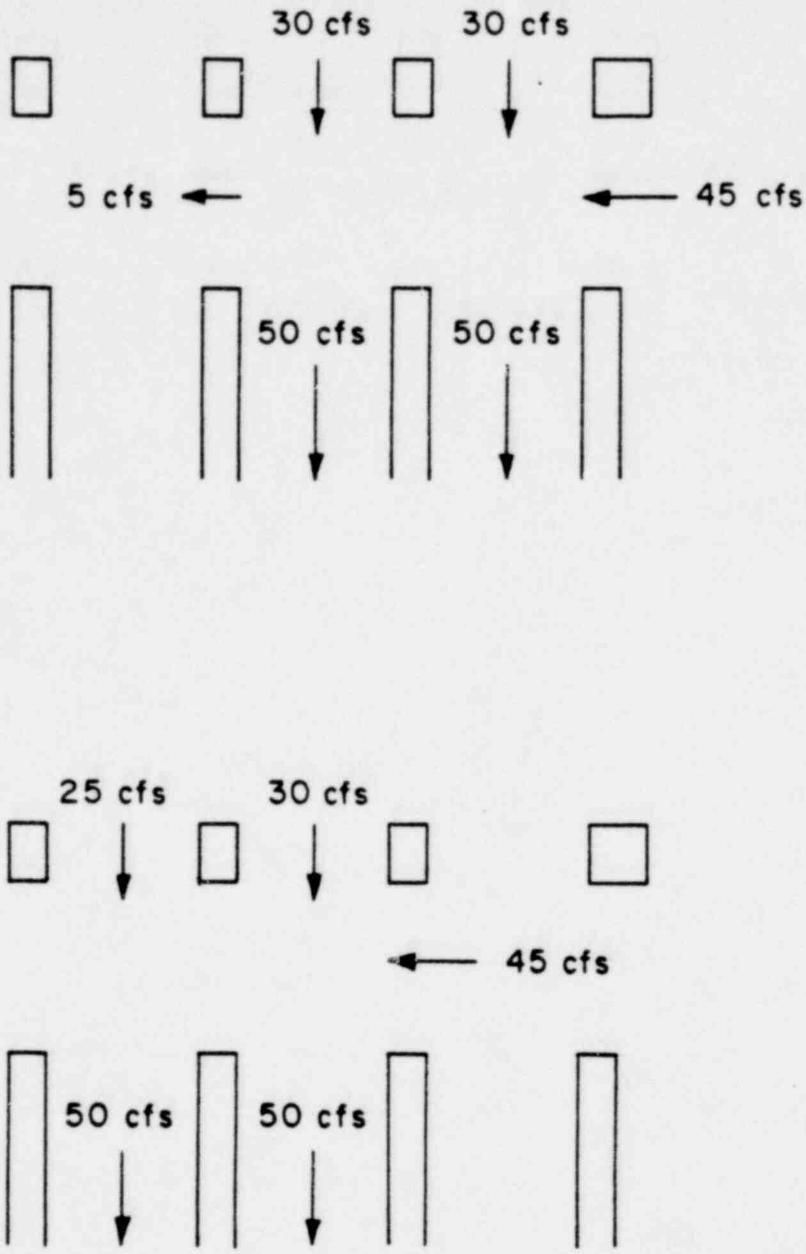

Table 10. Discharge distribution at intake structure no. 1 at the annually based 7-day 10-year low flow; stage = 582 ft

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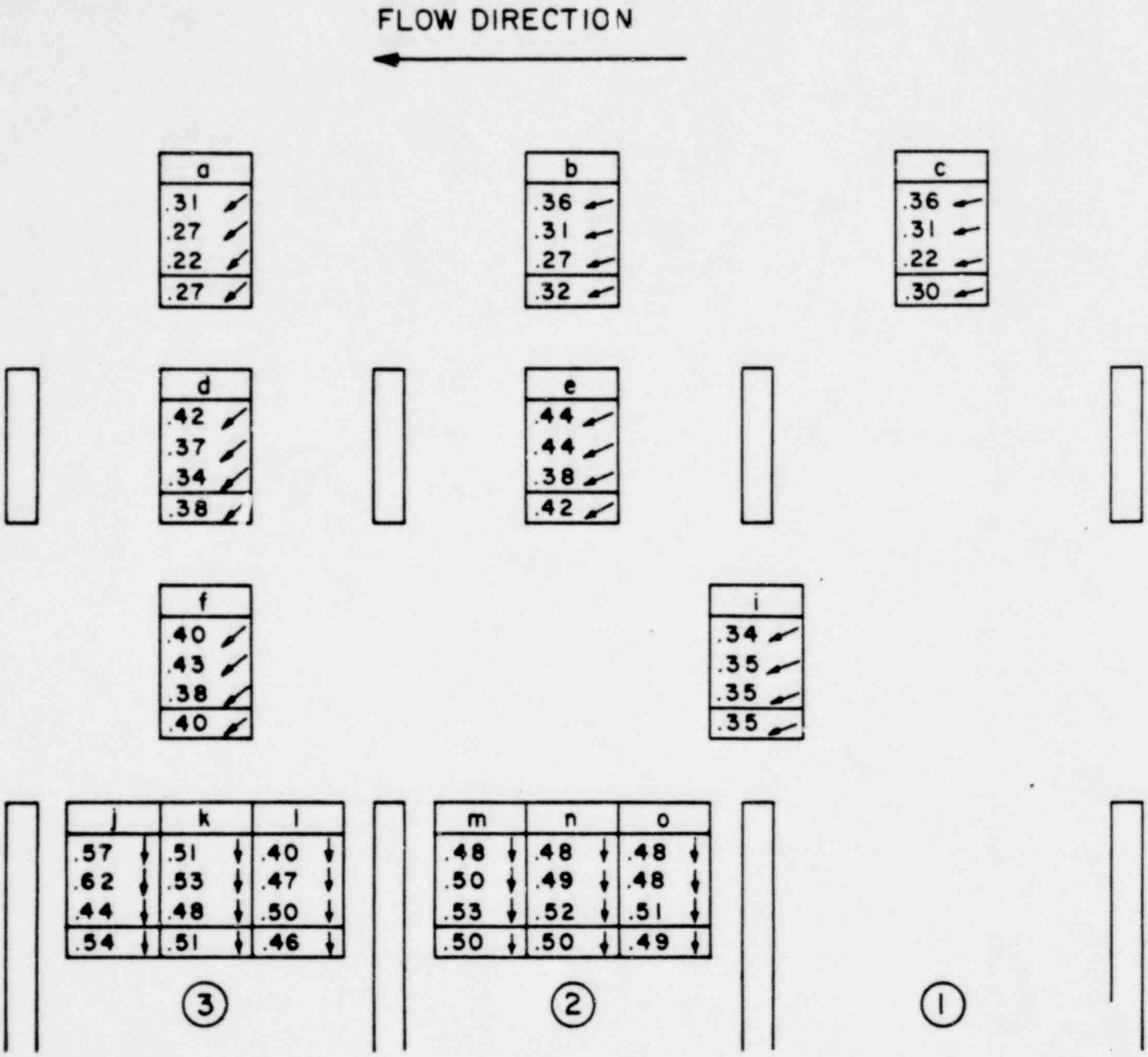

Table 12. Velocity measurements (in fps) at intake structure no. 2 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 2 and 3 in operation

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FLOW DIRECTION

←

(a)

Channel 3: (no measurements)

Channel 2:

m	n	o
.59 ↓	.50 ↓	.39 ↓
.63 ↓	.57 ↓	.42 ↓
.52 ↓	.50 ↓	.49 ↓
.58 ↓	.52 ↓	.43 ↓

Channel 1:

p	q	r
.46 ↓	.48 ↓	.47 ↓
.49 ↓	.50 ↓	.49 ↓
.53 ↓	.52 ↓	.50 ↓
.49 ↓	.50 ↓	.49 ↓

(a)

(b)

Channel 3:

j	k	l
.55 ↓	.49 ↓	.40 ↓
.61 ↓	.51 ↓	.44 ↓
.52 ↓	.49 ↓	.46 ↓
.56 ↓	.50 ↓	.43 ↓

Channel 2: (no measurements)

Channel 1:

p	q	r
.60 ↓	.44 ↓	.42 ↓
.56 ↓	.42 ↓	.50 ↓
.57 ↓	.45 ↓	.52 ↓
.58 ↓	.44 ↓	.48 ↓

(b)

(c)

Channel 3:

j	k	l
.53 ↓	.45 ↓	.48 ↓
.52 ↓	.45 ↓	.50 ↓
.57 ↓	.52 ↓	.49 ↓
.54 ↓	.47 ↓	.49 ↓

Channel 2: sealed off by stop logs according to design

Channel 1:

p	q	r
.53 ↓	.59 ↓	.36 ↓
.52 ↓	.55 ↓	.53 ↓
.42 ↓	.48 ↓	.53 ↓
.49 ↓	.54 ↓	.47 ↓

(c)

Table 13. Velocity measurements (in fps) across the intake channels of intake structure no. 2 at the 7-day 10-year low flow for April and May; stage = 583 ft; (a) pumps 1 and 2 in operation; (b) and (c) pumps 1 and 3 in operation

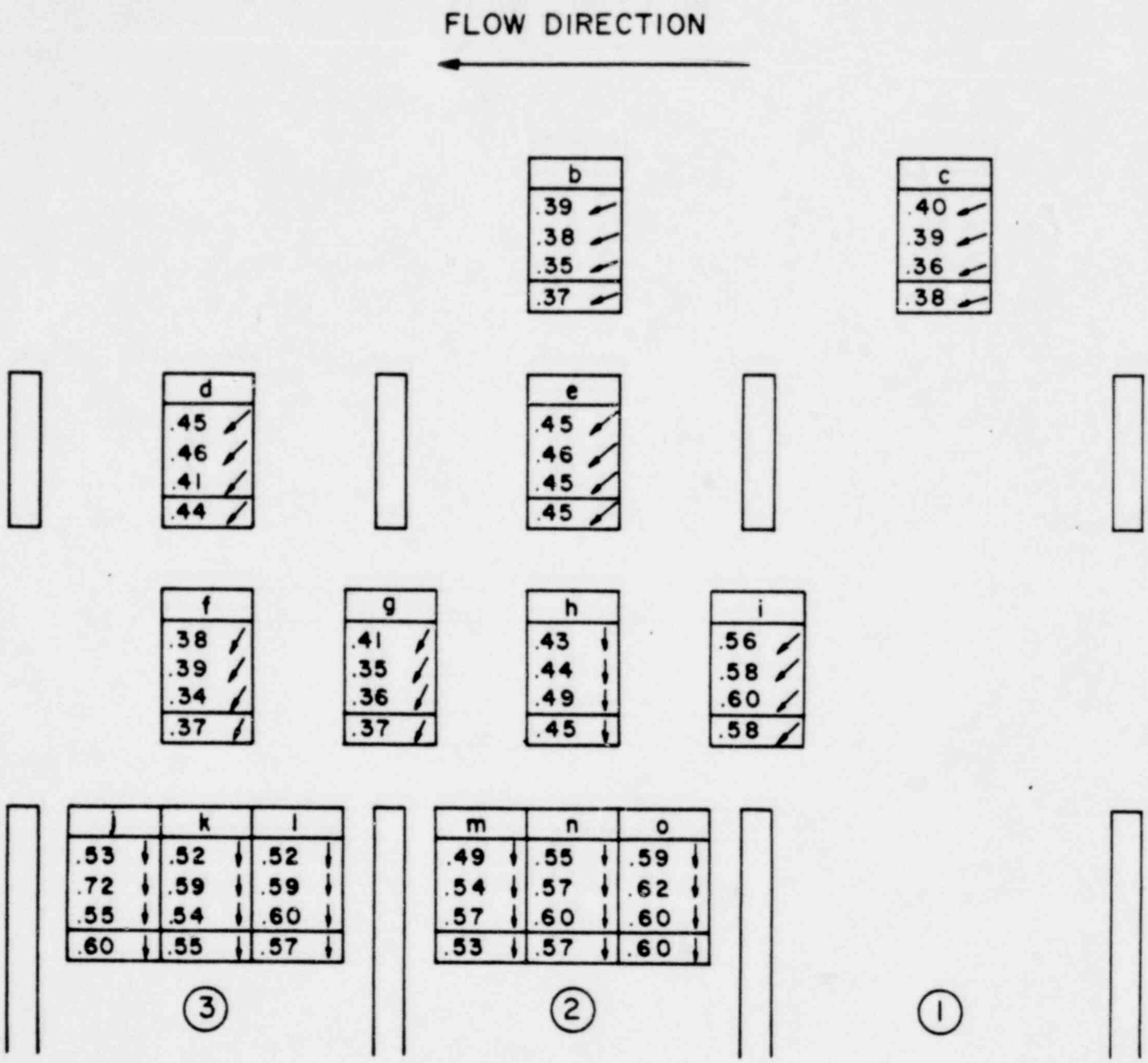

Table 14. Velocity measurements (in fps) at intake structure no. 2 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 2 and 3 in operation

2330 268

FLOW DIRECTION

←

m	n	o
.62 ↓	.55 ↓	.49 ↓
.71 ↓	.59 ↓	.58 ↓
.59 ↓	.54 ↓	.58 ↓
.64 ↓	.56 ↓	.55 ↓

p	q	r
.48 ↓	.55 ↓	.51 ↓
.54 ↓	.60 ↓	.62 ↓
.59 ↓	.60 ↓	.61 ↓
.54 ↓	.58 ↓	.58 ↓

3 2 1

Table 15. Velocity measurements (in fps) across the intake channels of intake structure no. 2 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 1 and 2 in operation

2330 269

Distance upstream from nozzle (ft)	Distance from shoreline (ft)	Depth of water, d (ft)	Velocity (fps) at level:			Depth-average velocity (fps)
			0.8d	0.4d	0.2d	
75	150	14.0	.48	.42	.36	.42
			.46	.42	.36	.42 *
	125	13.8	.48	.42	.38	.43
			.47	.42	.36	.42 *
	100	12.3	.48	.41	.35	.41
			.48	.43	.37	.43 *
25	150	14.0	.54	.48	.41	.48
			.48	.44	.39	.44 *
	125	13.0	.53	.47	.42	.47
			.46	.42	.38	.42 *

* without discharge

Table 16. Velocity measurements upstream from discharge nozzle at the annually based 7-day 10-year low flow; stage = 582 ft

2330 270

WITHDRAWAL CONDITION	CHANNEL 1	CHANNEL 2	CHANNEL 3
I. 7-day, 10-yr low flow for April and May			
a. Pumps 1 & 2 operating	0.42-0.52	0.33-0.49	--
b. Pumps 2 & 3 operating	--	0.33-0.56	0.36-0.45
II. 7-day, 10-yr low flow annual basis			
a. Pumps 1 & 2 operating	0.37-0.57	0.35-0.48	--
b. Pumps 2 & 3 operating	--	0.38-0.63	0.37-0.52

Table 17. Range of depth-averaged intake velocities (fps) across the intake channels of structure no. 1

2330 271

Figure 1. The area modeled

2330 272

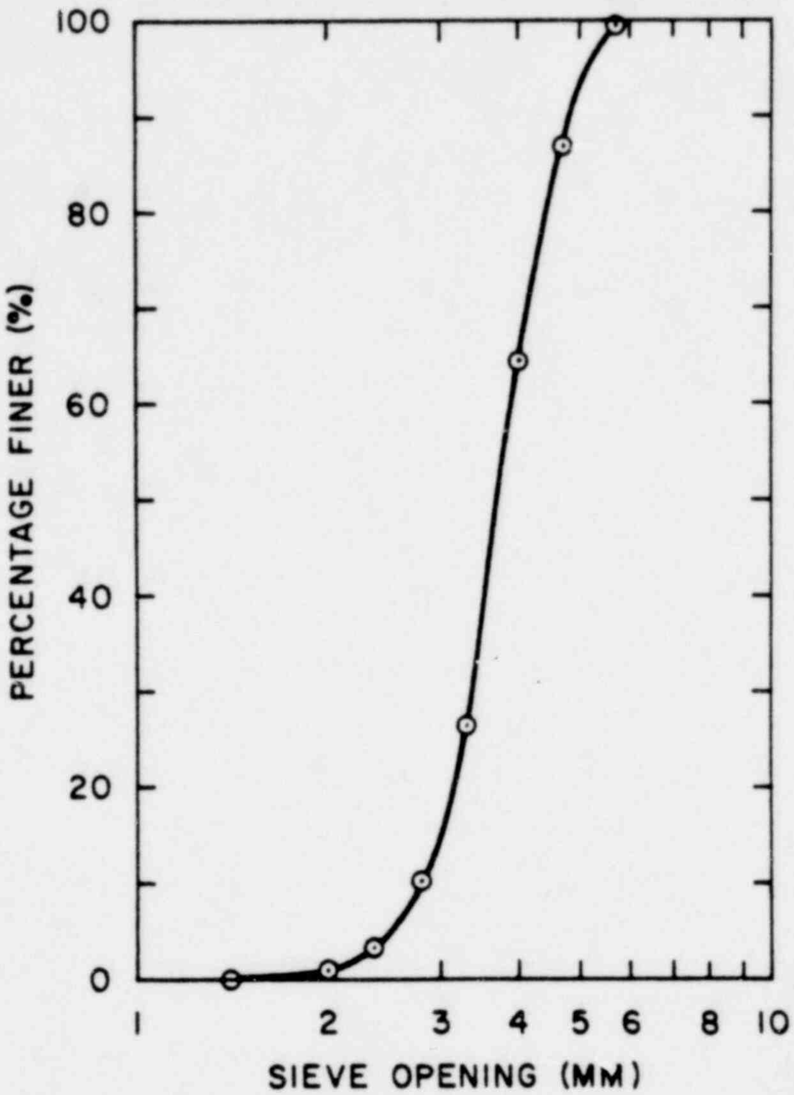

Figure 2. Distribution of sieve diameters of model sediment

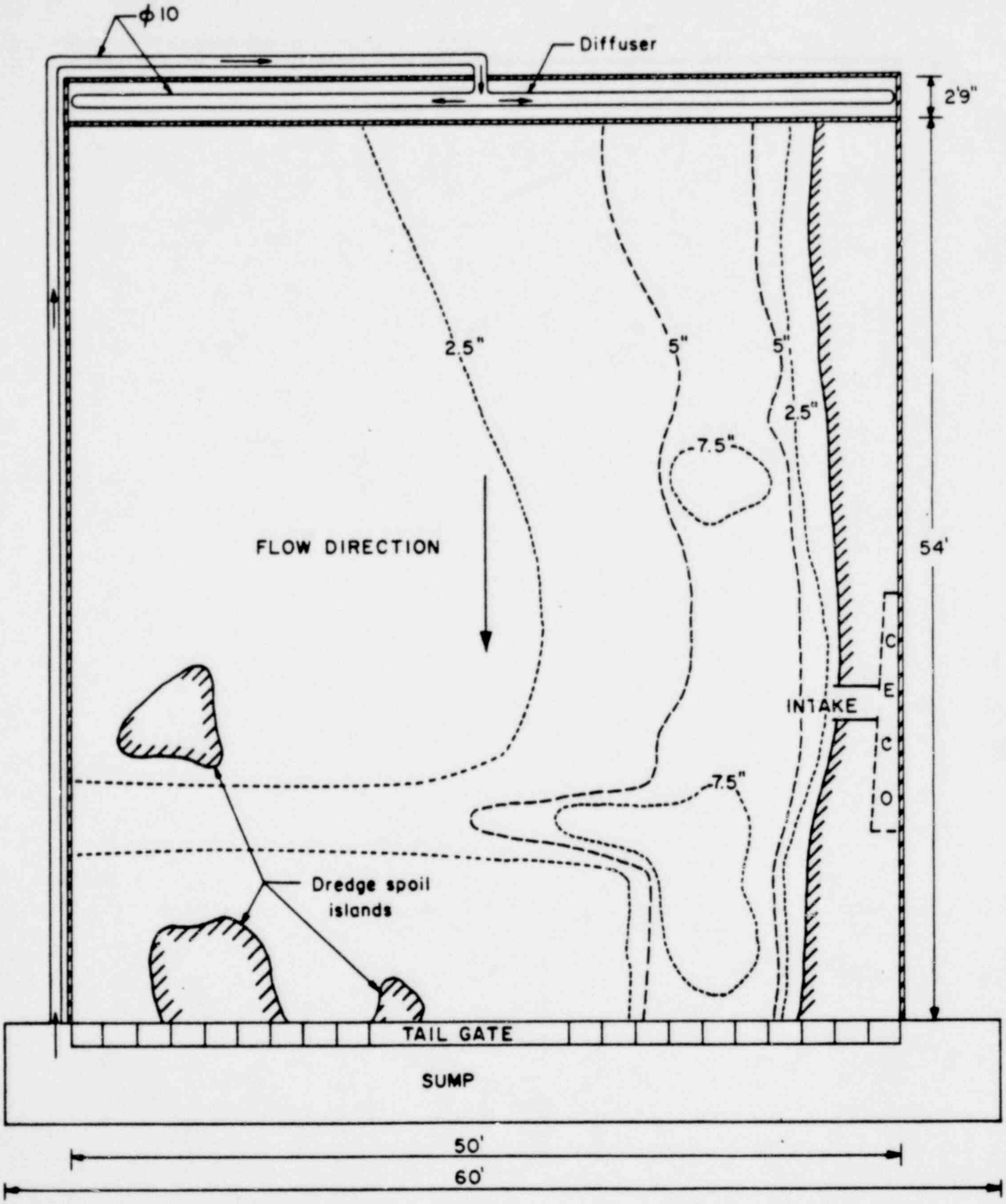

Figure 3. General layout of model

2330 274

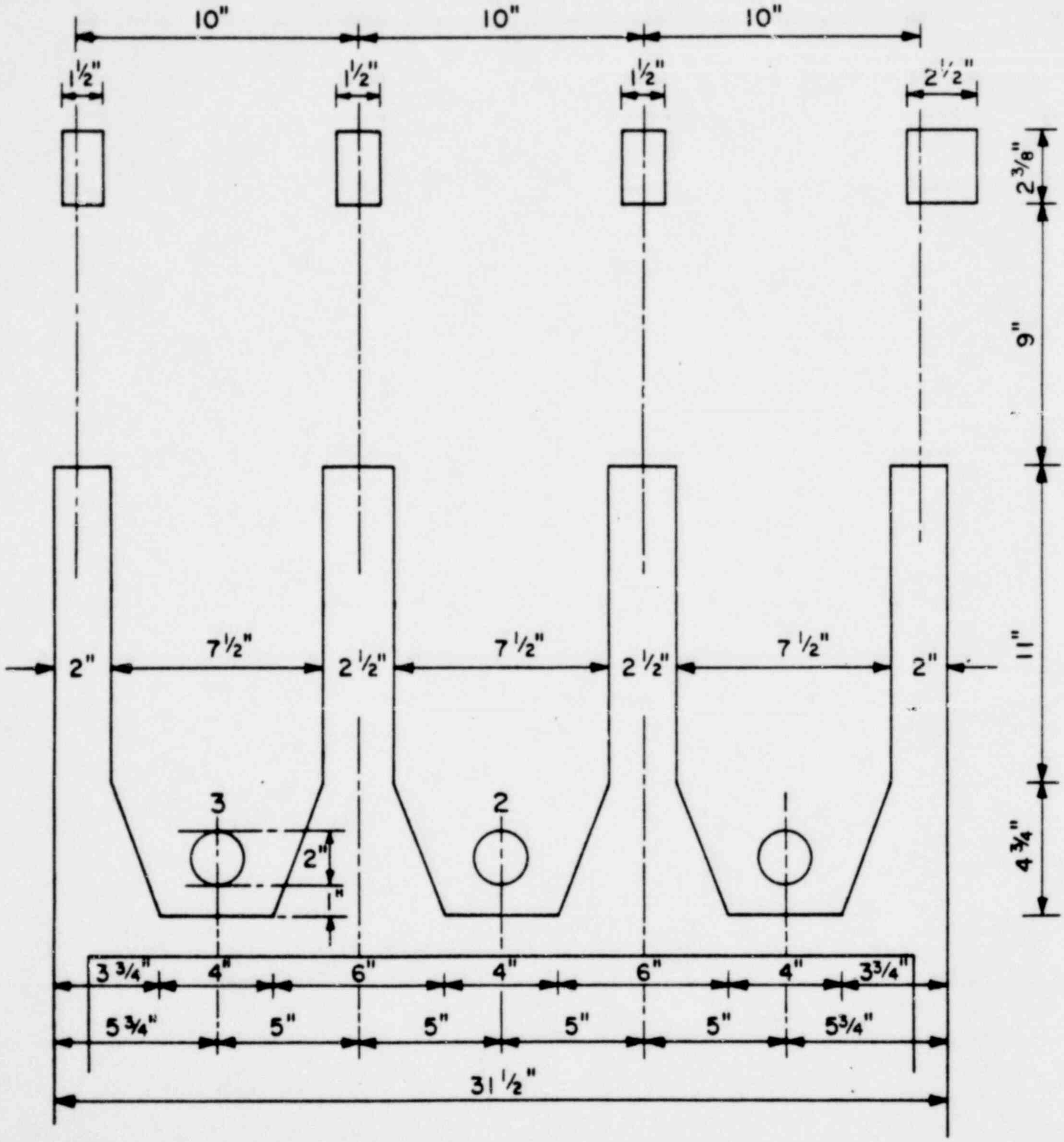

Figure 4. Intake structure no. 1 modeled after SL's drawing no. M-16.
Scale ratio = 1:24

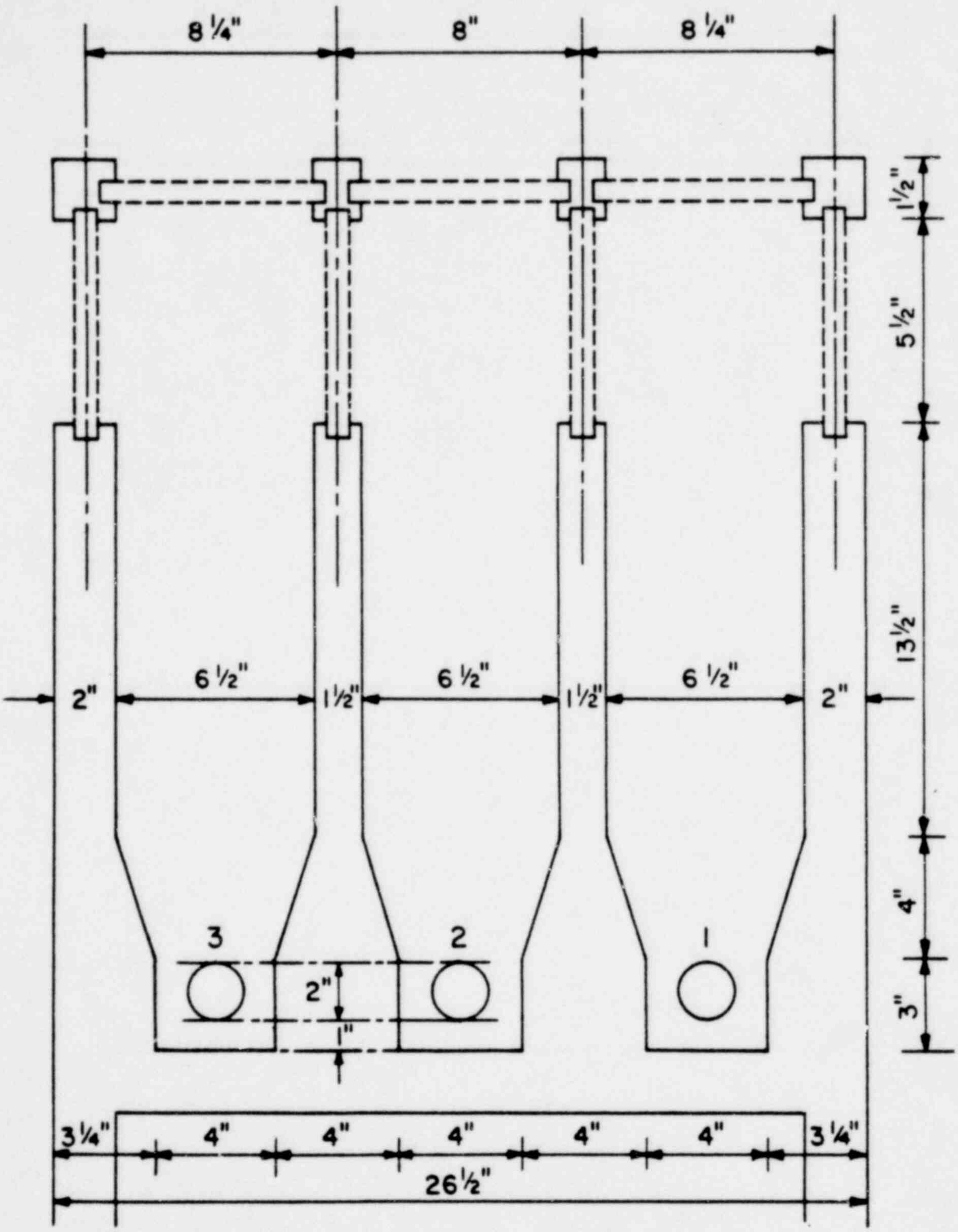

Figure 5. Intake structure no. 2 modeled after SL's drawing no. MS-10. Scale ratio = 1:24

2330 276

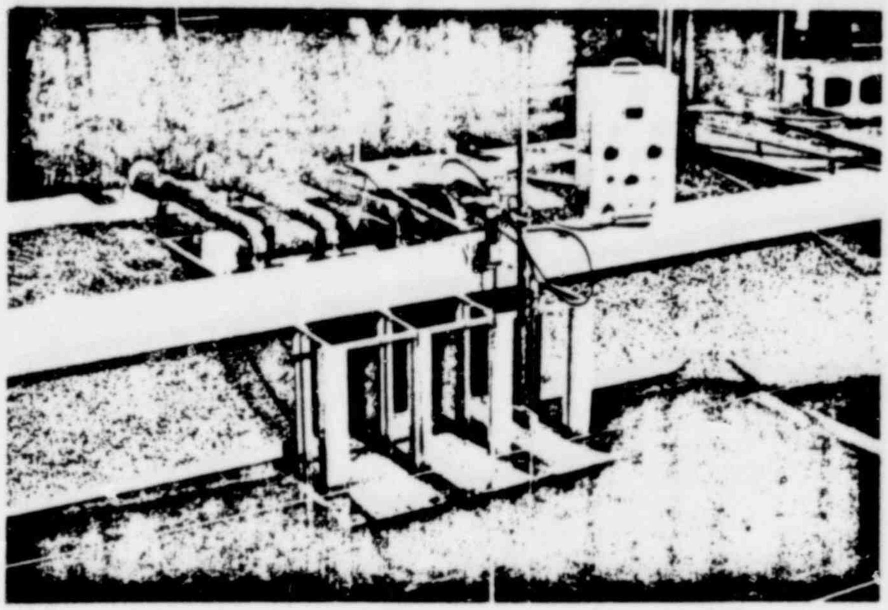

Figure 6. Set-up of current meter for velocity measurements

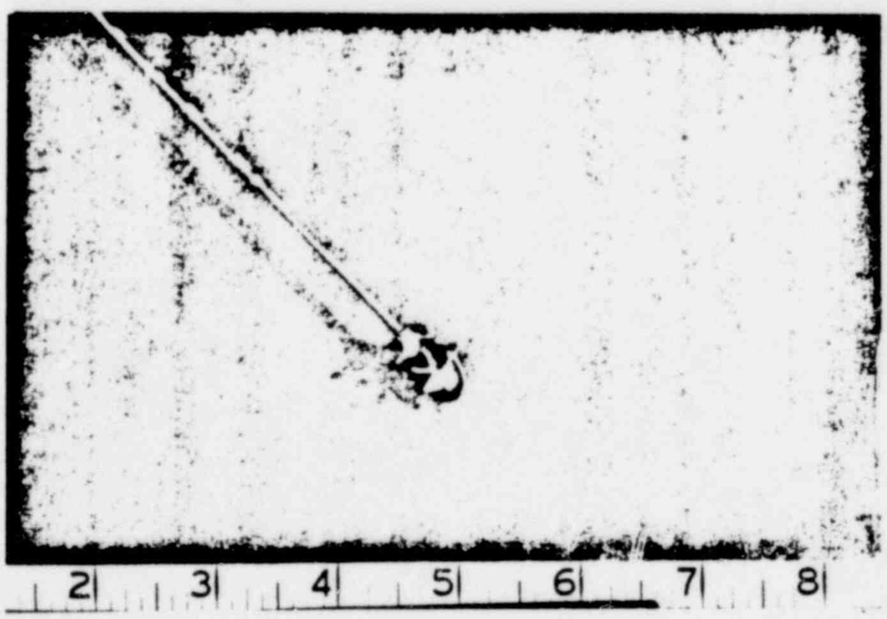

Figure 7. Close-up of current meter probe

2330 277

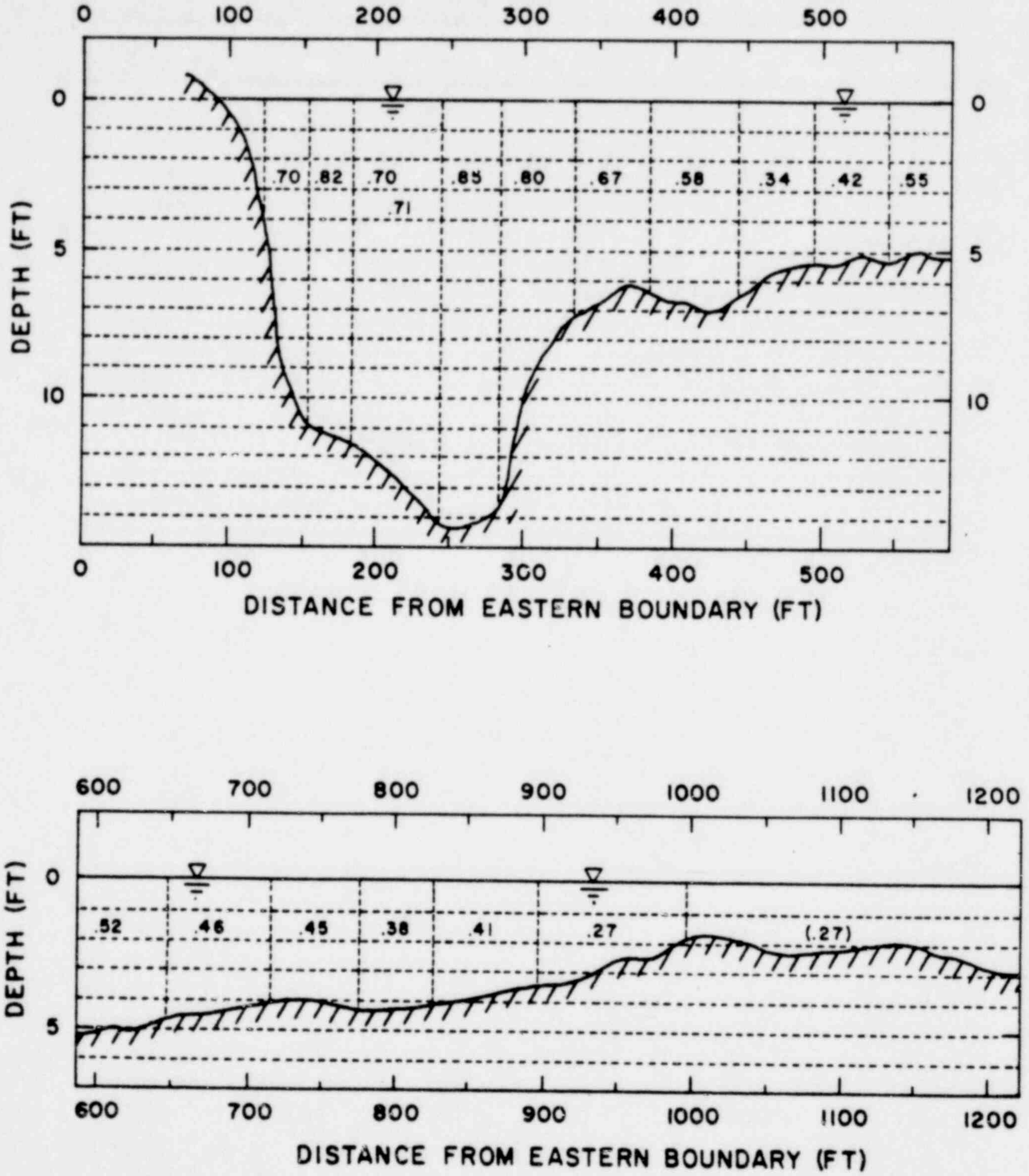

Figure 8. Velocity measurements in the prototype along transect no. 7 on 12. October 1977

2330 278

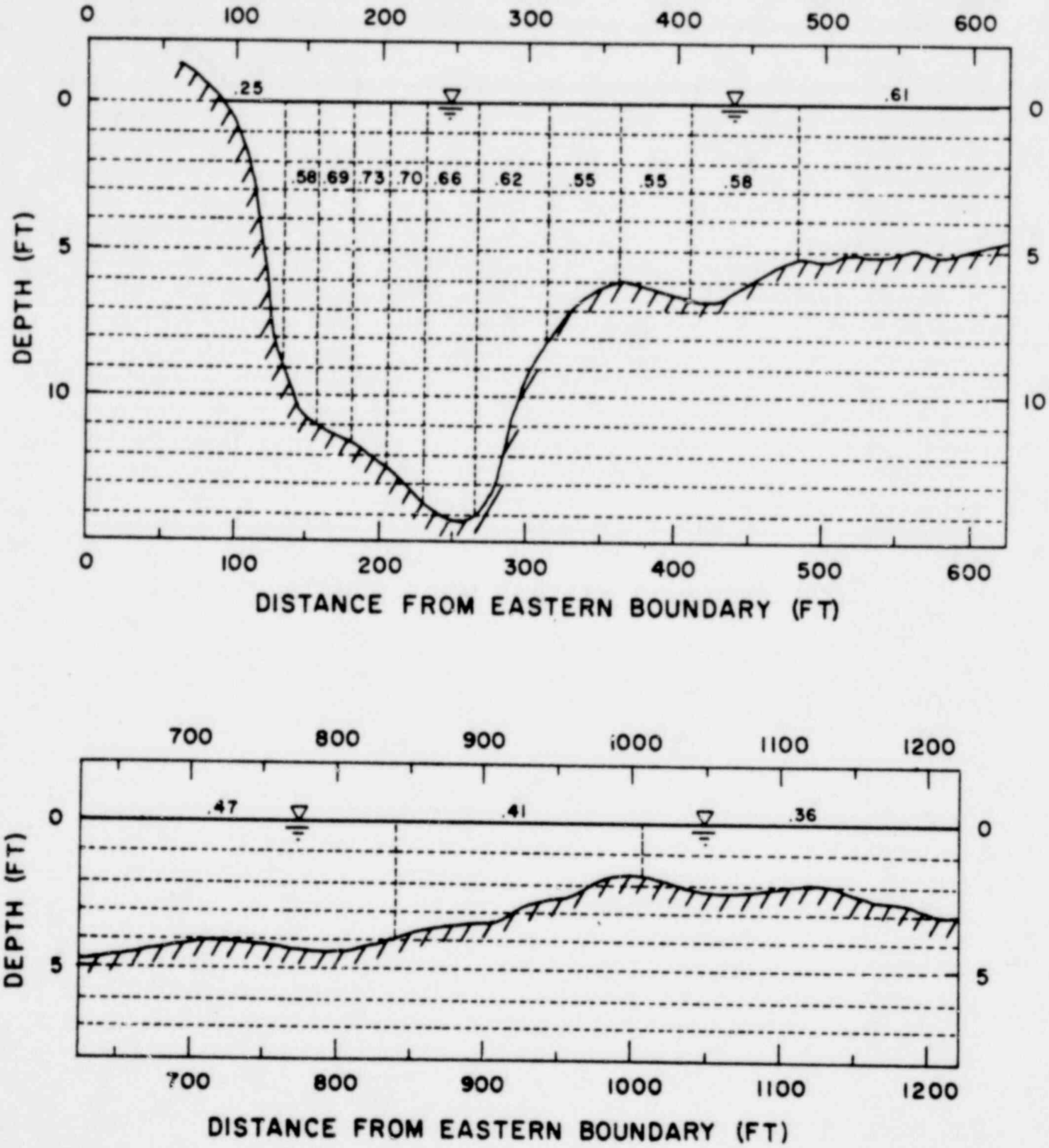

Figure 9. Velocity measurements along transects no. 7 in the calibration flow of the model. Velocities listed below the surface of the water are depth averaged velocities; velocities listed above the surface of the water are float velocities

2330 279

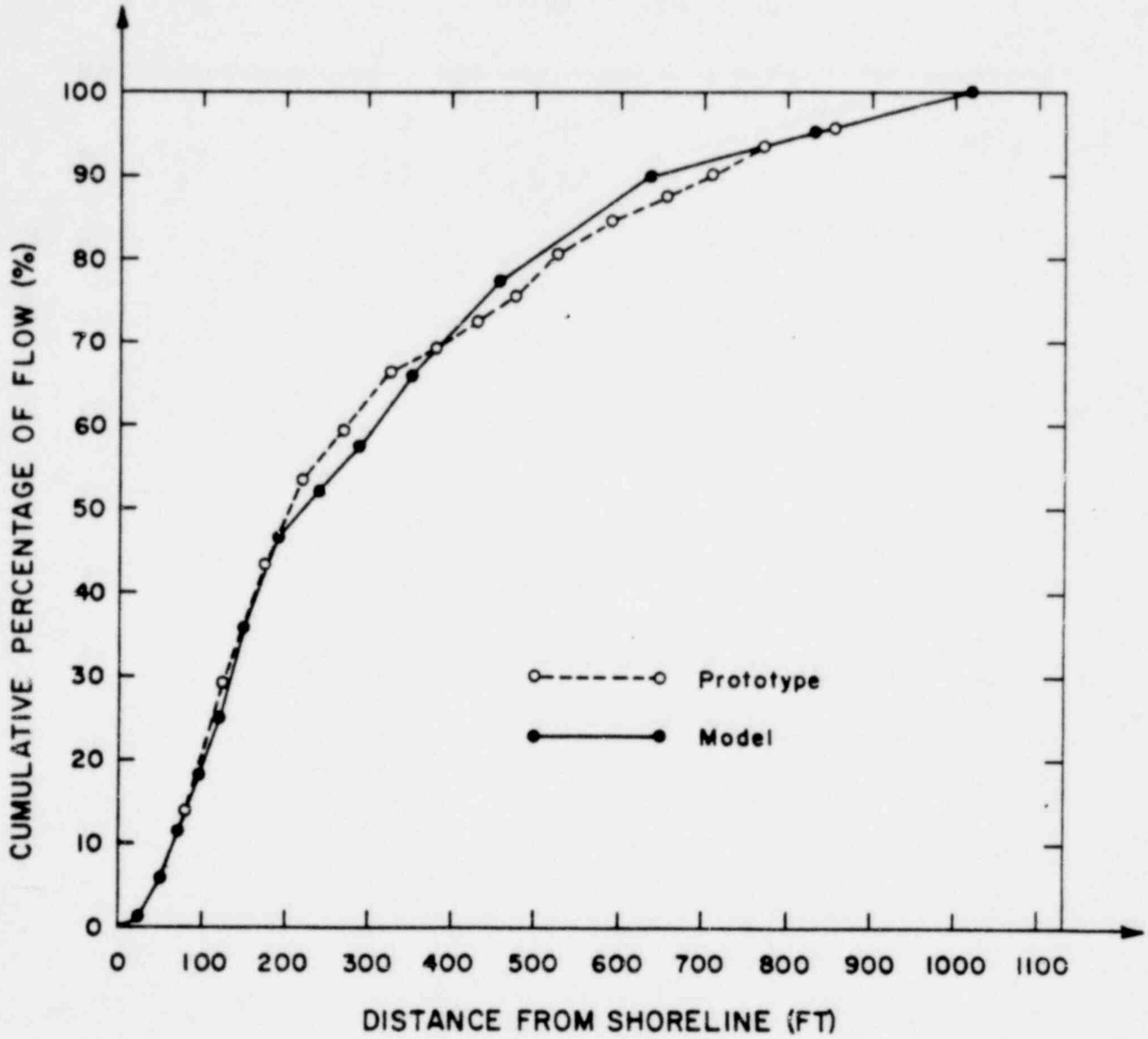

Figure 10 Distribution of cumulative percentage of flow through section no. 7 in the calibration flow

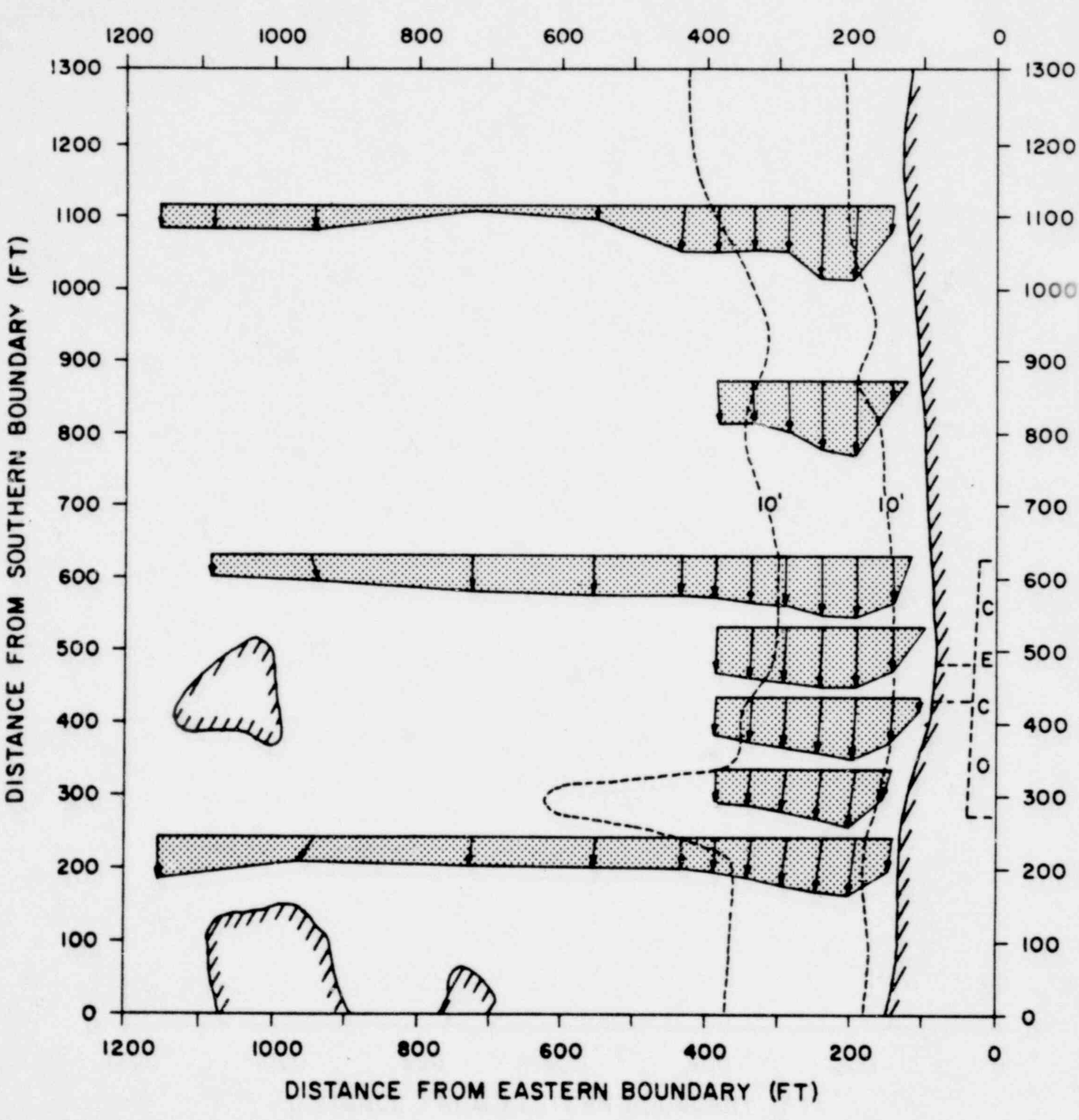

Figure 11. Velocity distributions in the calibration flow of the model

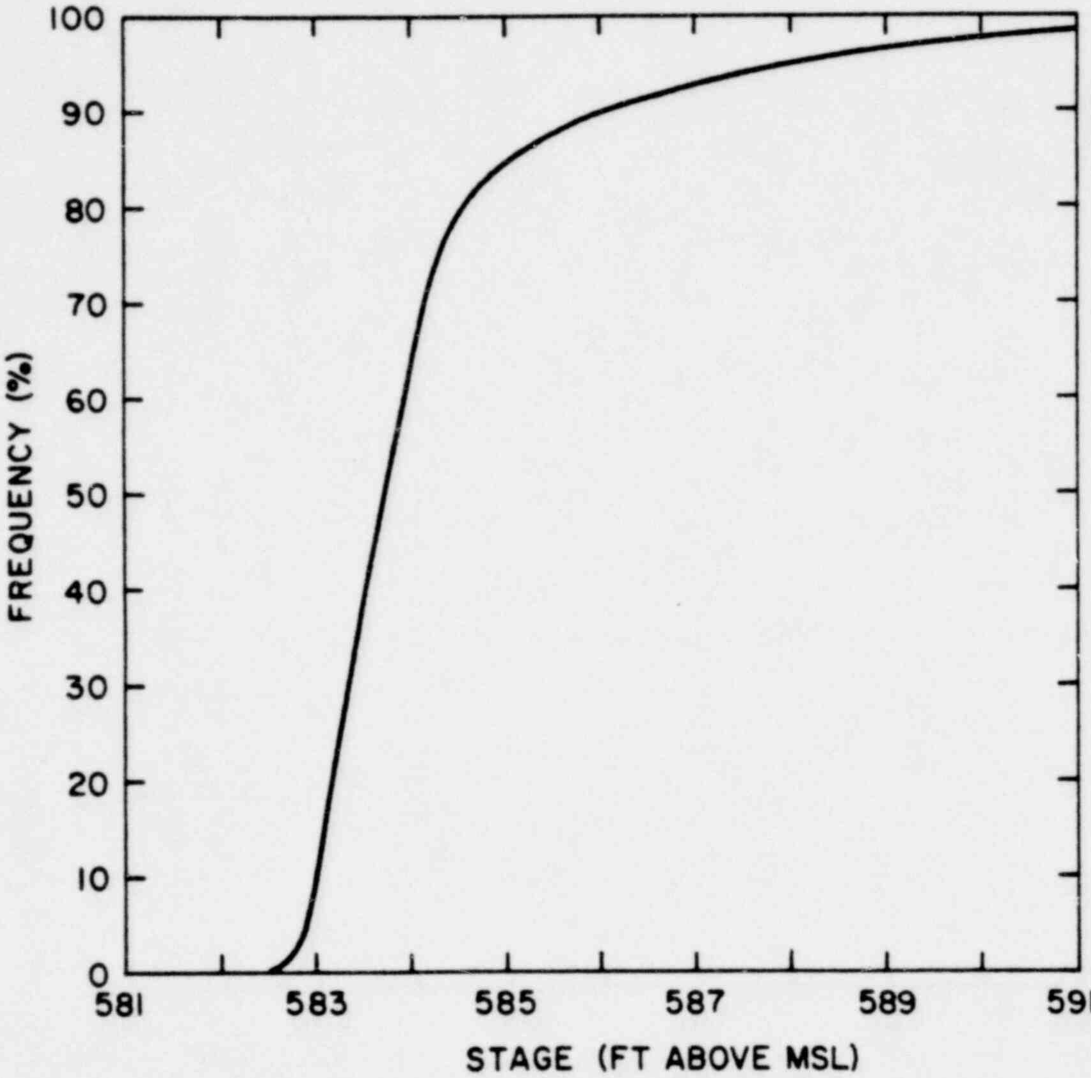

Figure 12. Cumulative frequency distribution of the stage at the proposed position of the intake (at River Mile 530.2) for April and May; based on daily means, 1947-77.

2330 282

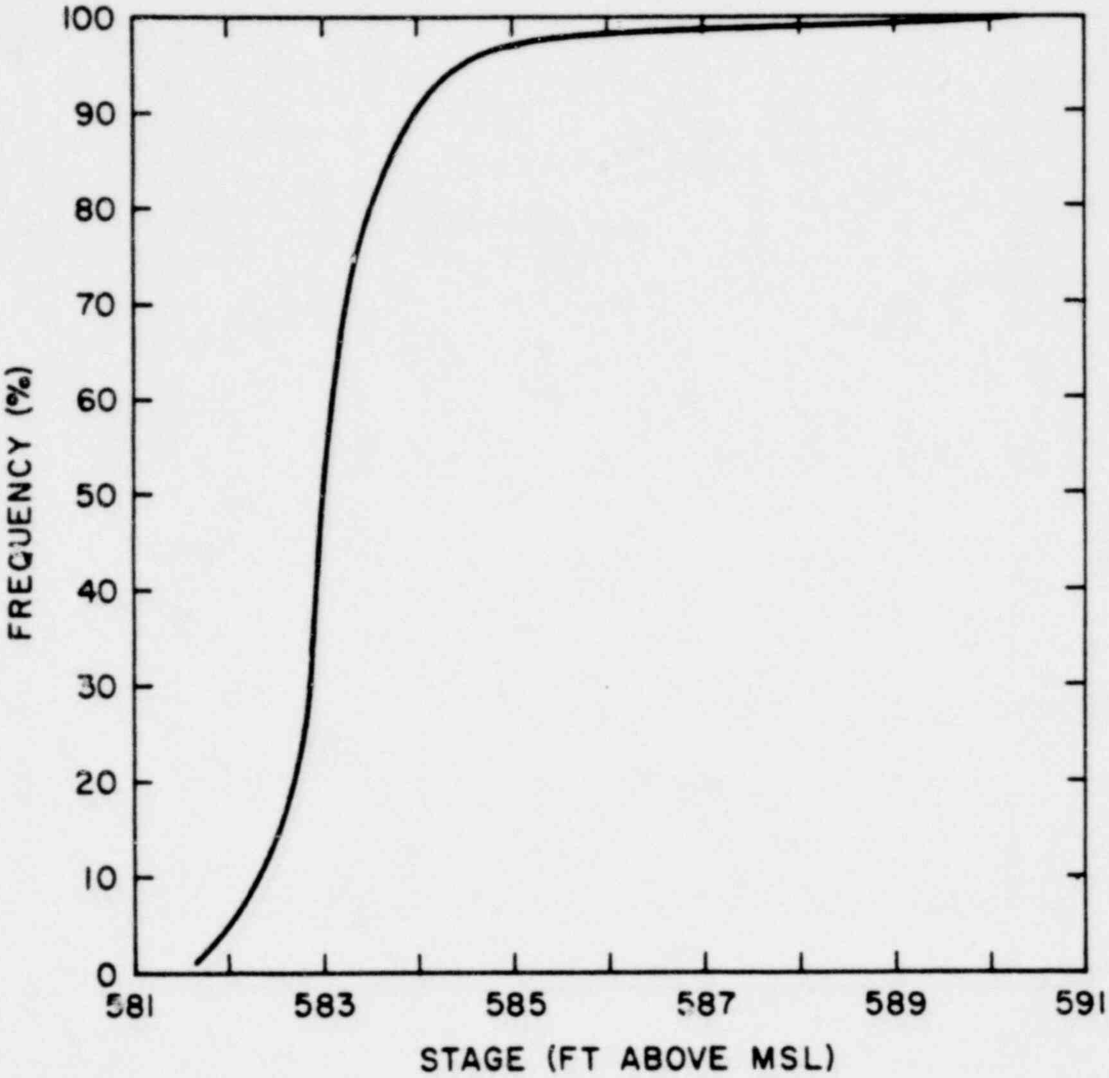

Figure 13. Cumulative frequency distribution of the stage at the proposed position of the intake (at River Mile 530.2); based on daily means, 1947-77.

2330 283

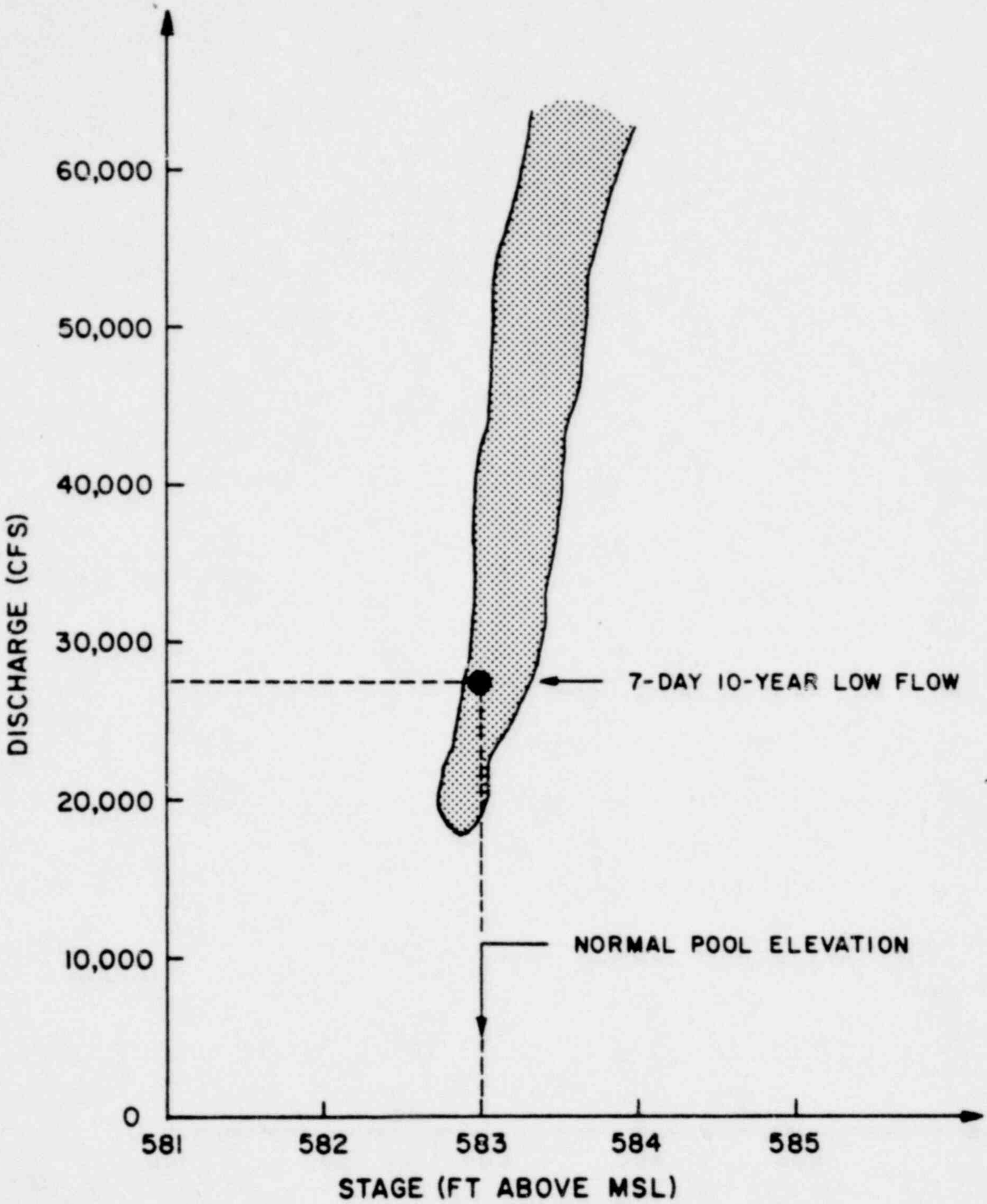

Figure 14. Correlation between discharge and stage at the proposed position of the intake (at River Mile 530.2) for April and May; based on daily means, 1947-77.

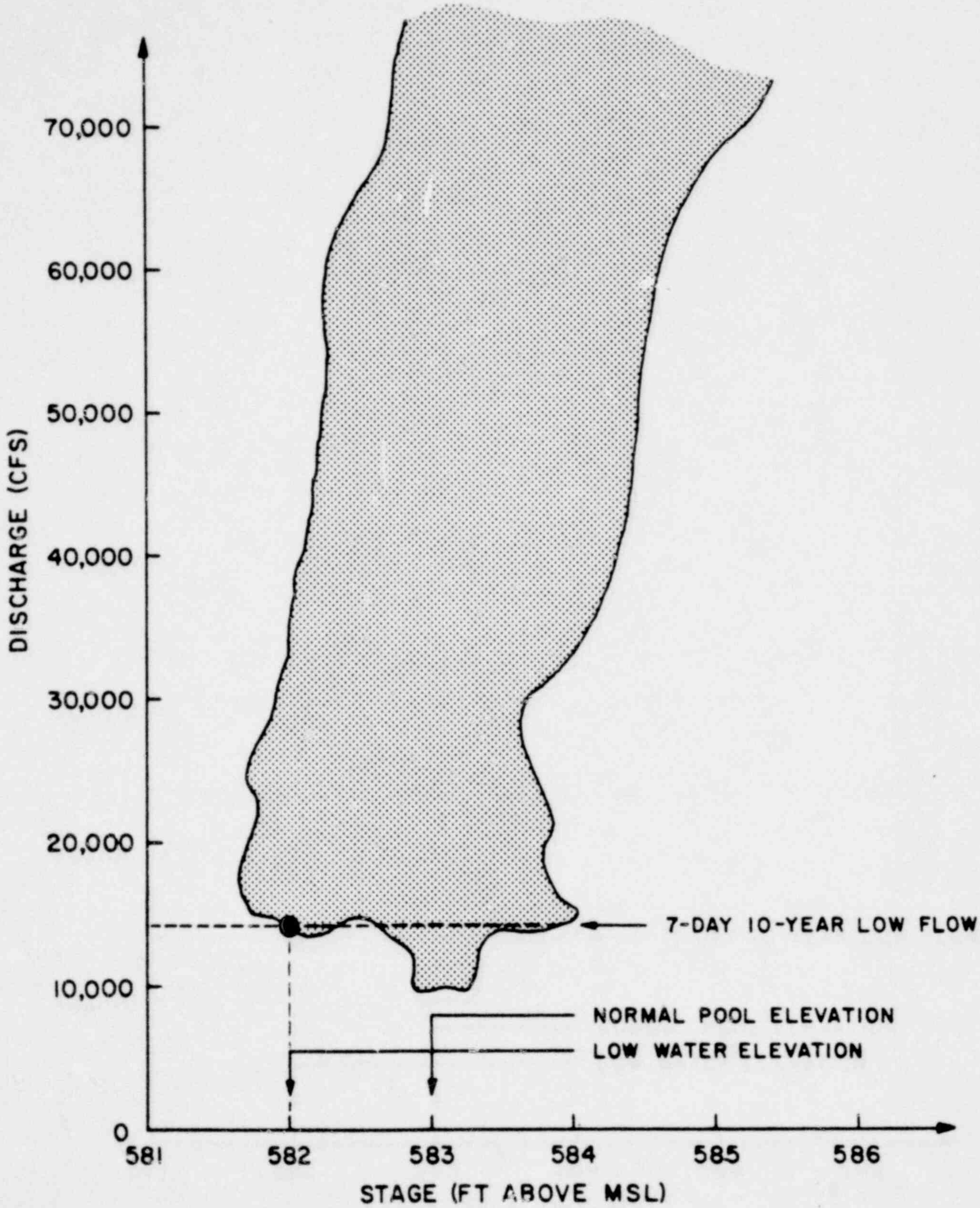

Figure 15. Correlation between discharge and stage at the proposed position of the intake (at River Mile 530.2); based on daily means, 1947-77.

2330 285

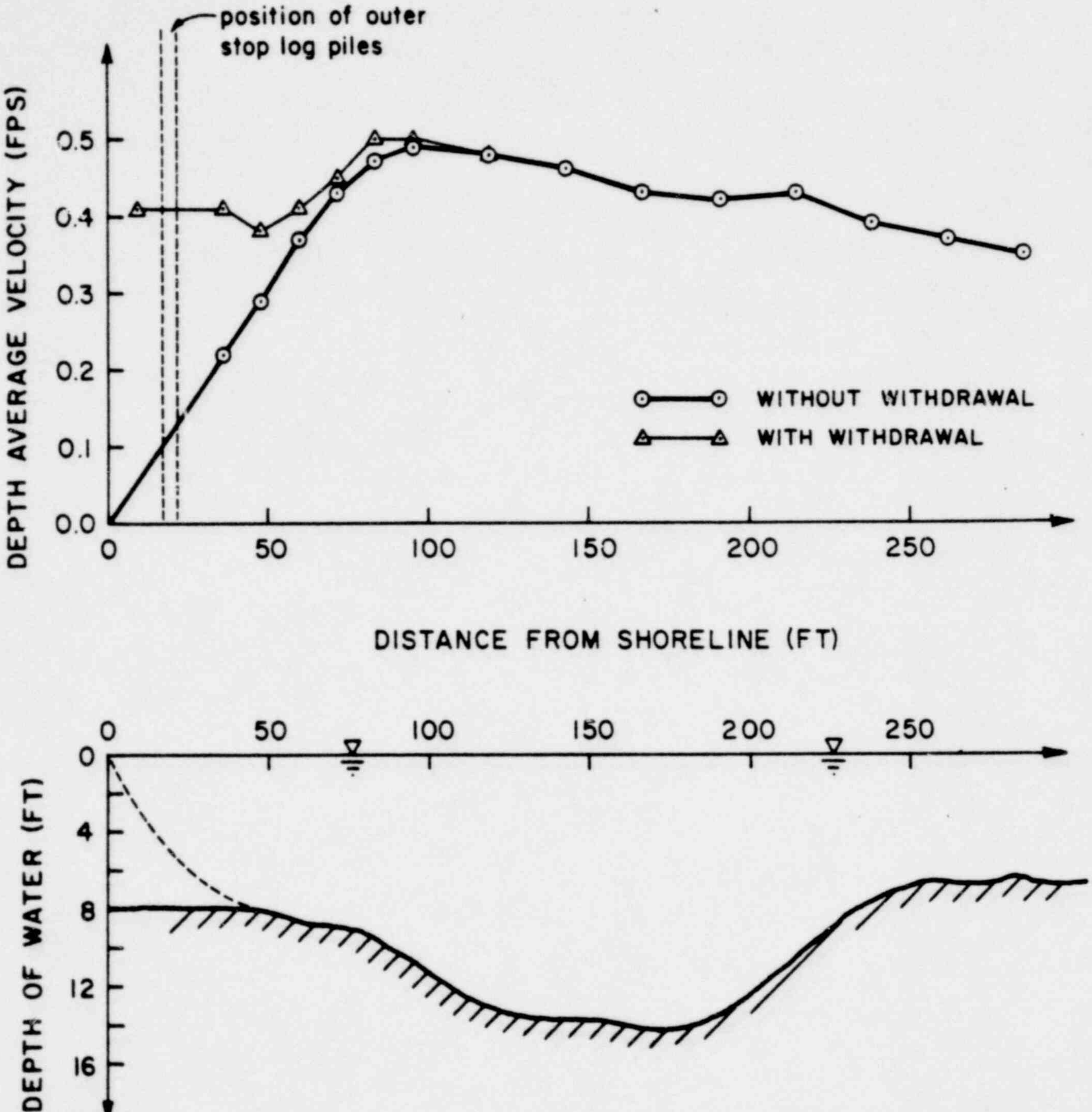

Figure 16. Distribution of the depth averaged velocity across the near-shore channel at the intake (no.1) at the 7-day 10-year low flow for April and May; stage = 583 ft

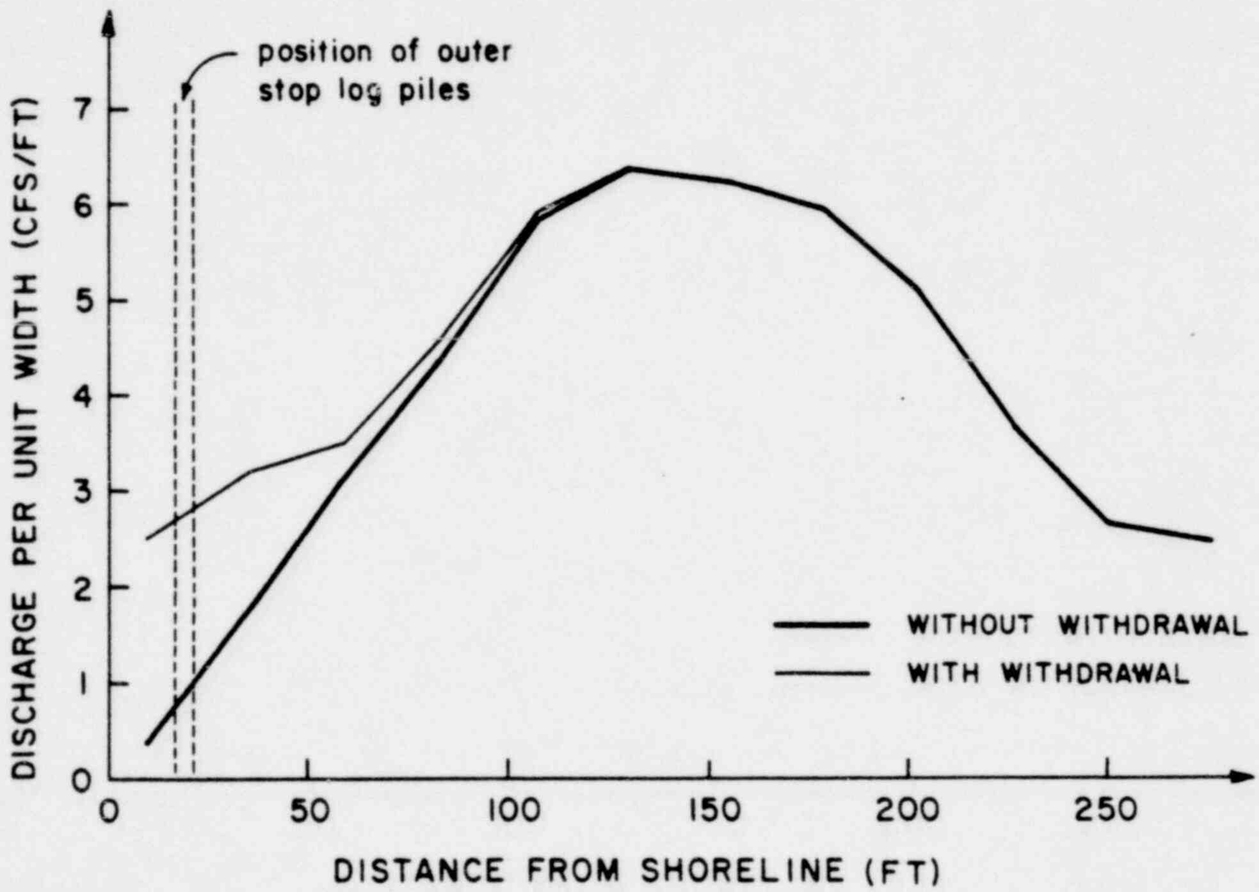

Figure 17. Distribution of the discharge per unit width across the nearshore channel at the intake at the 7-day 10-year low flow for April and May; stage = 583 ft

2330 287

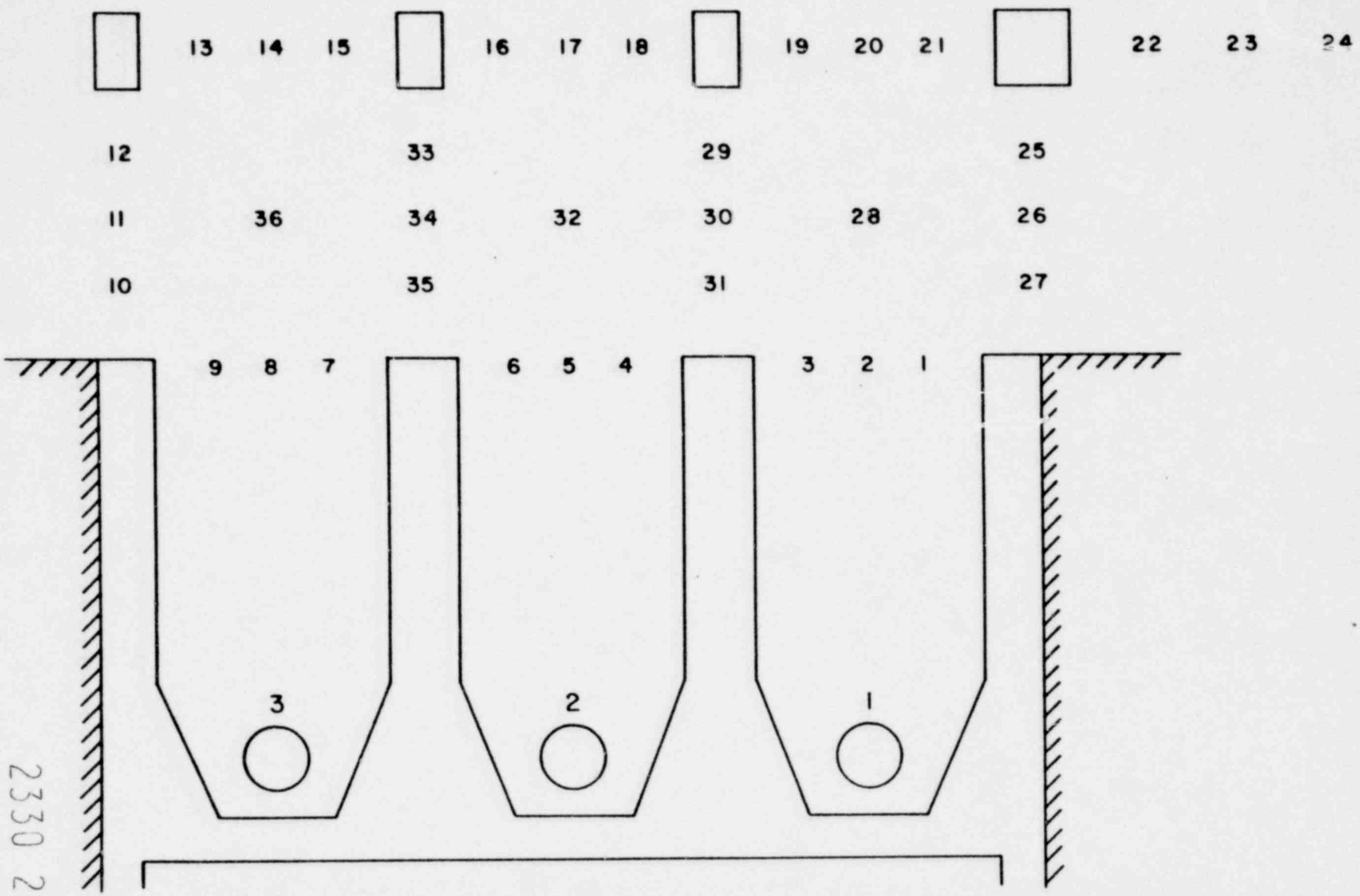

Figure 18. Points of measurement at intake structure no. 1

2330 288

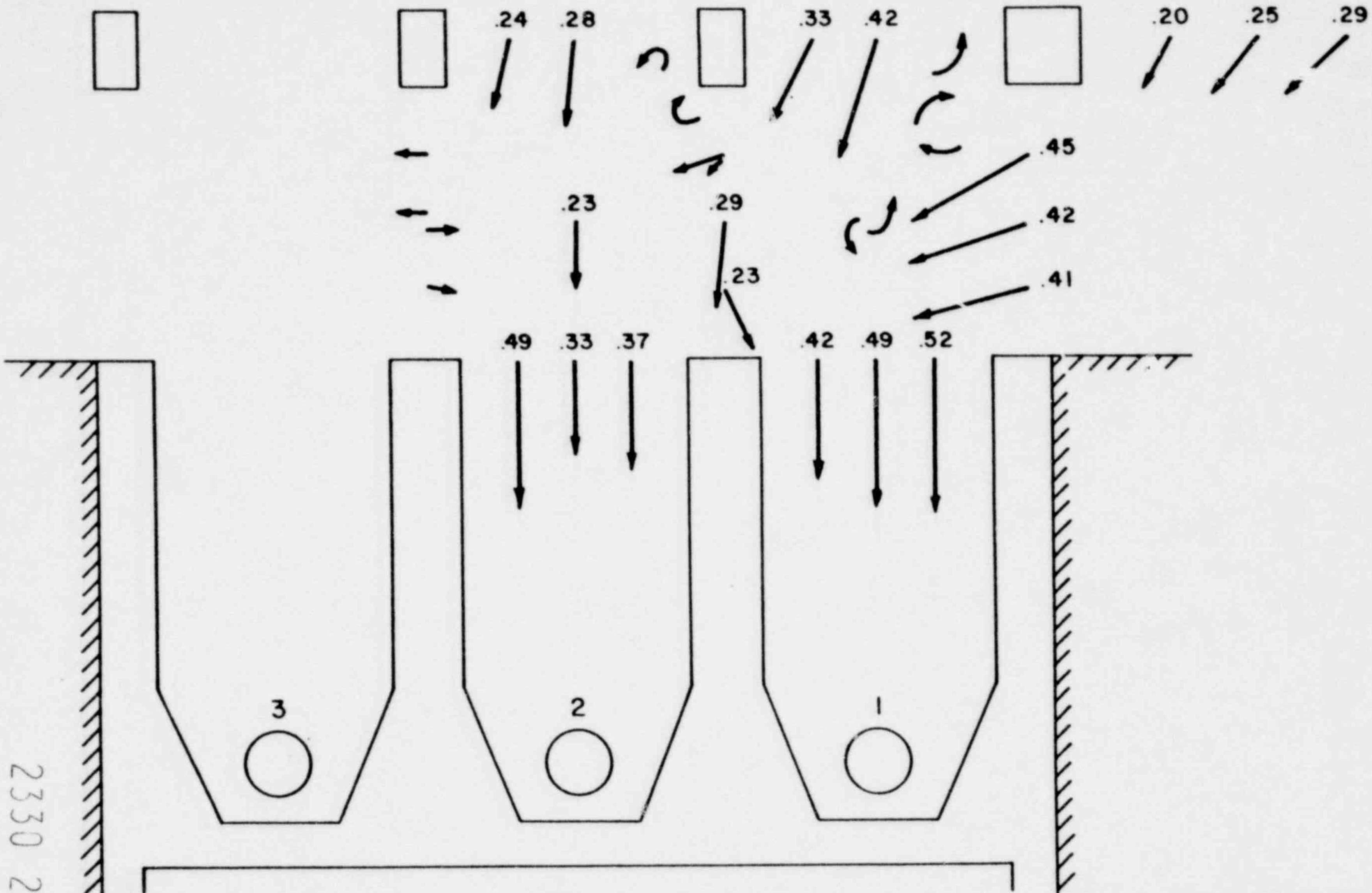

Figure 19. Flow pattern at intake structure no. 1 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 1 and 2 in operation

2330 289

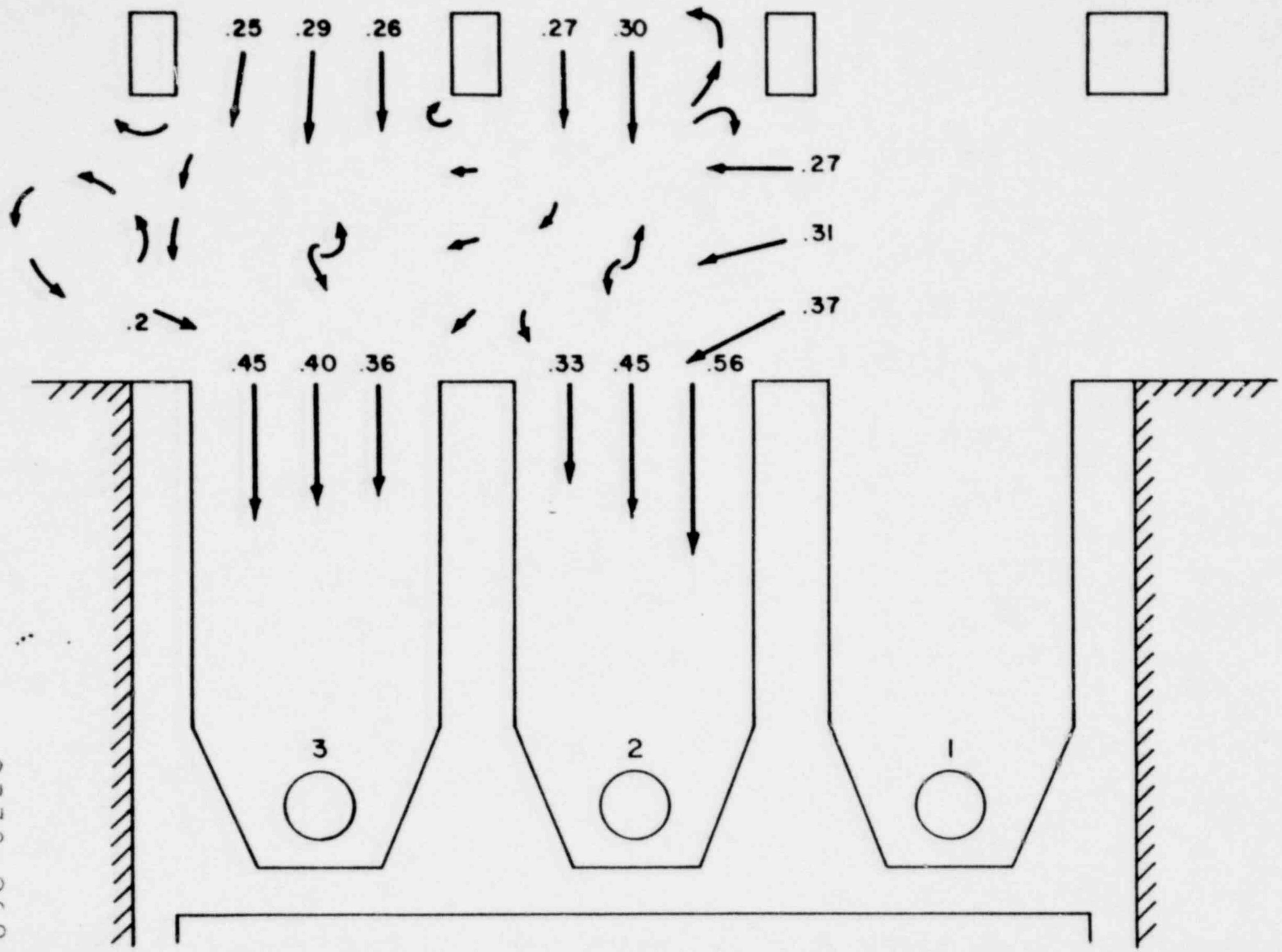

Figure 20. Flow pattern at intake structure no. 1 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 2 and 3 in operation.

2330 270

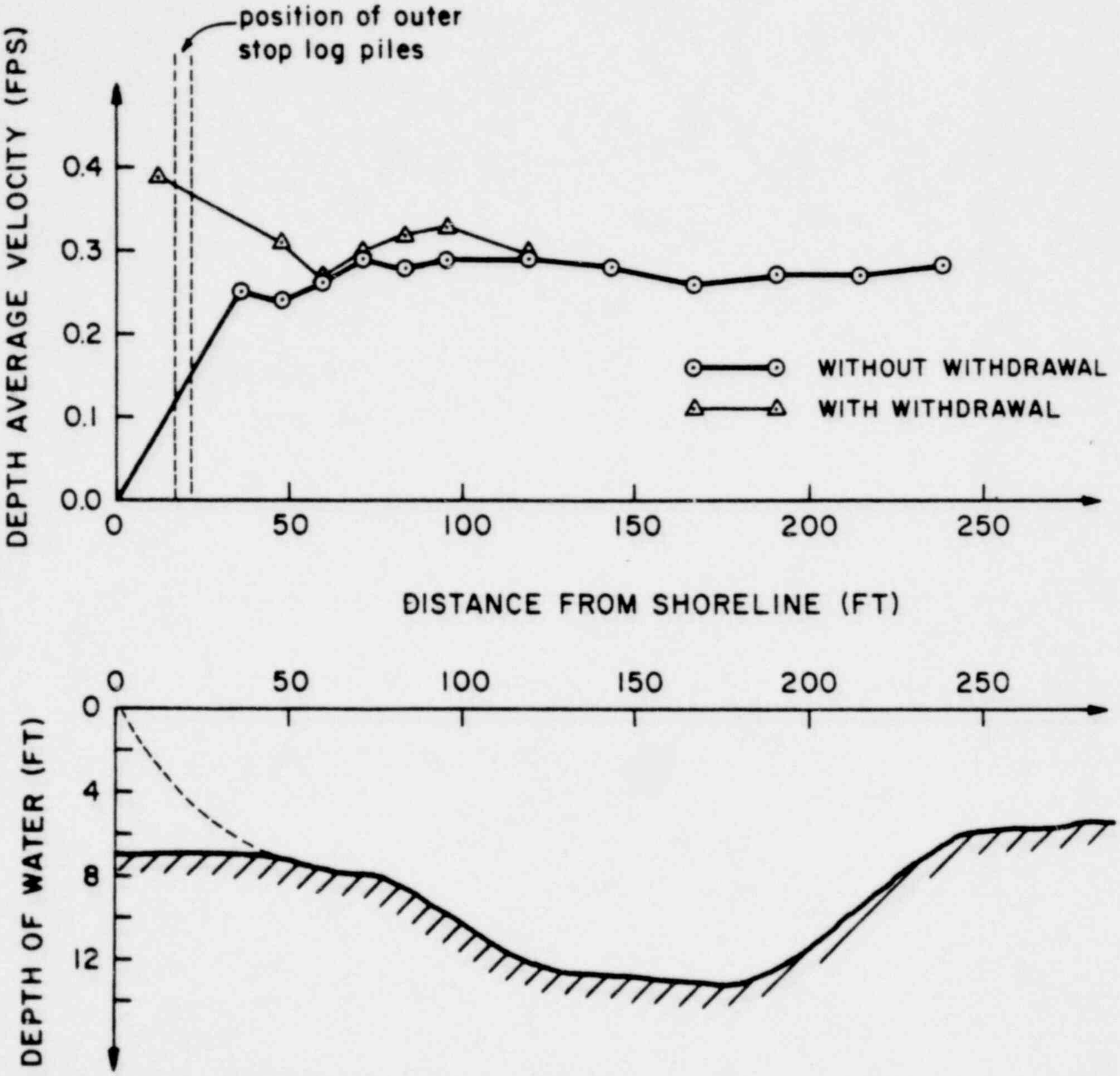

Figure 21. Distribution of the depth averaged velocity across the nearshore channel at the intake at the annually based 7-day 10-year low flow; stage = 582 ft.

2330 291

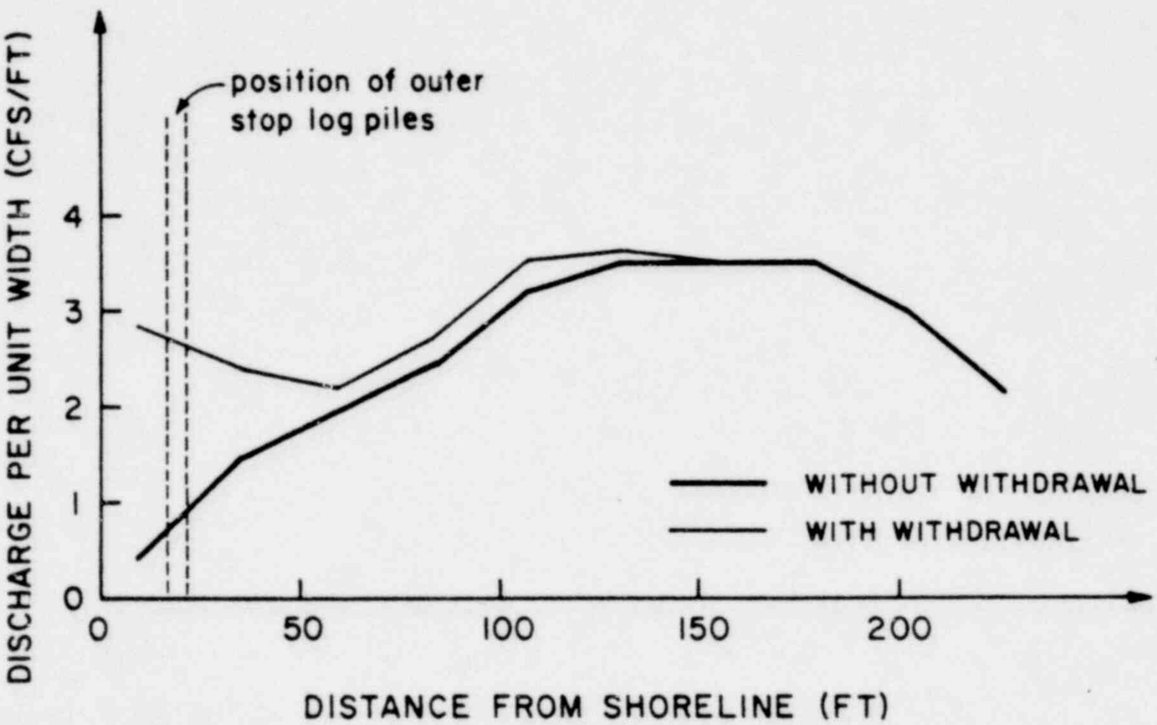

Figure 22. Distribution of the discharge per unit width across the nearshore channel at the intake at the annually based 7-day 10-year low flow; stage = 582 ft

2330 292

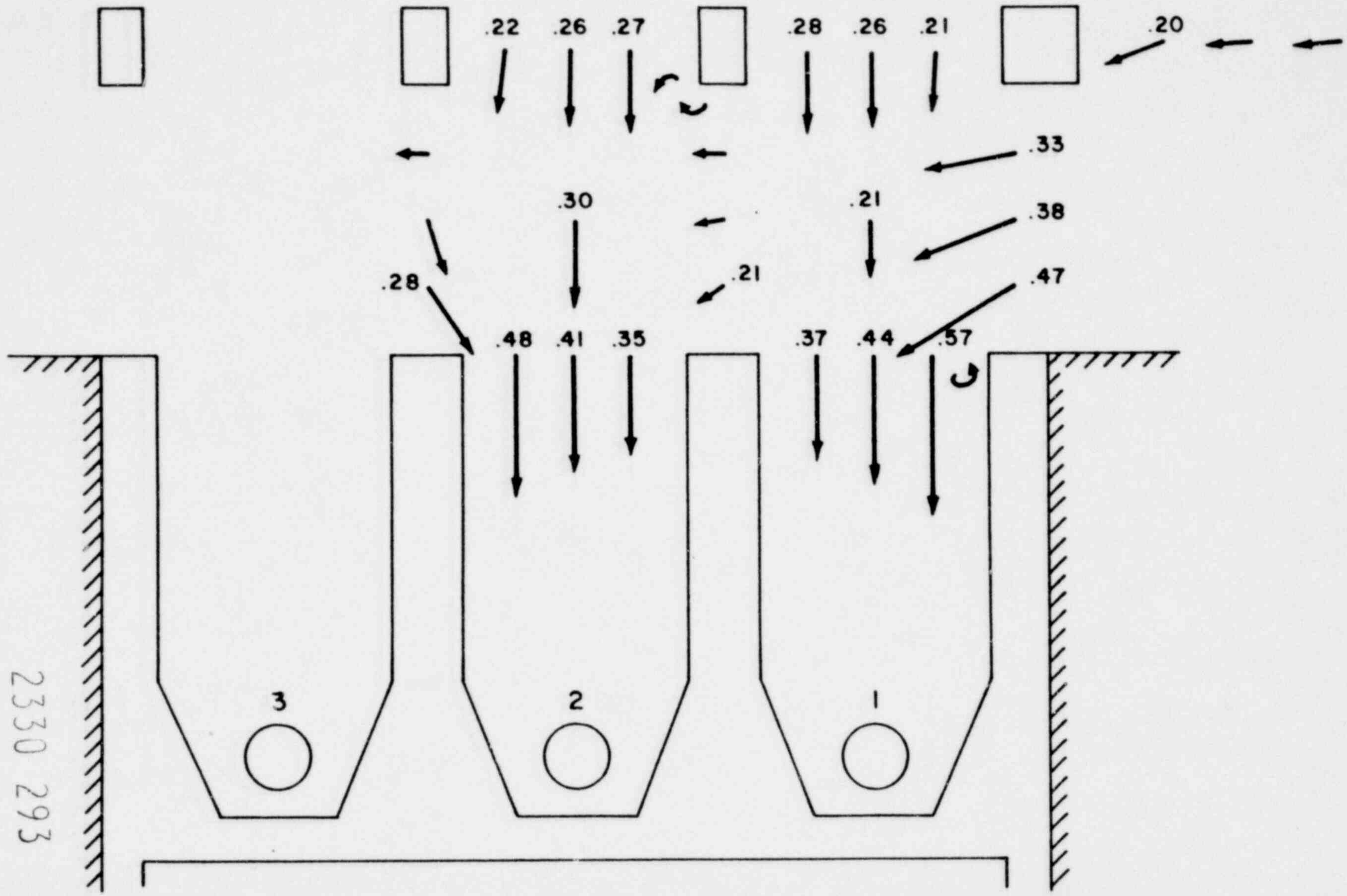

Figure 23. Flow pattern at intake structure no. 1 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 1 and 2 in operation

2330 293

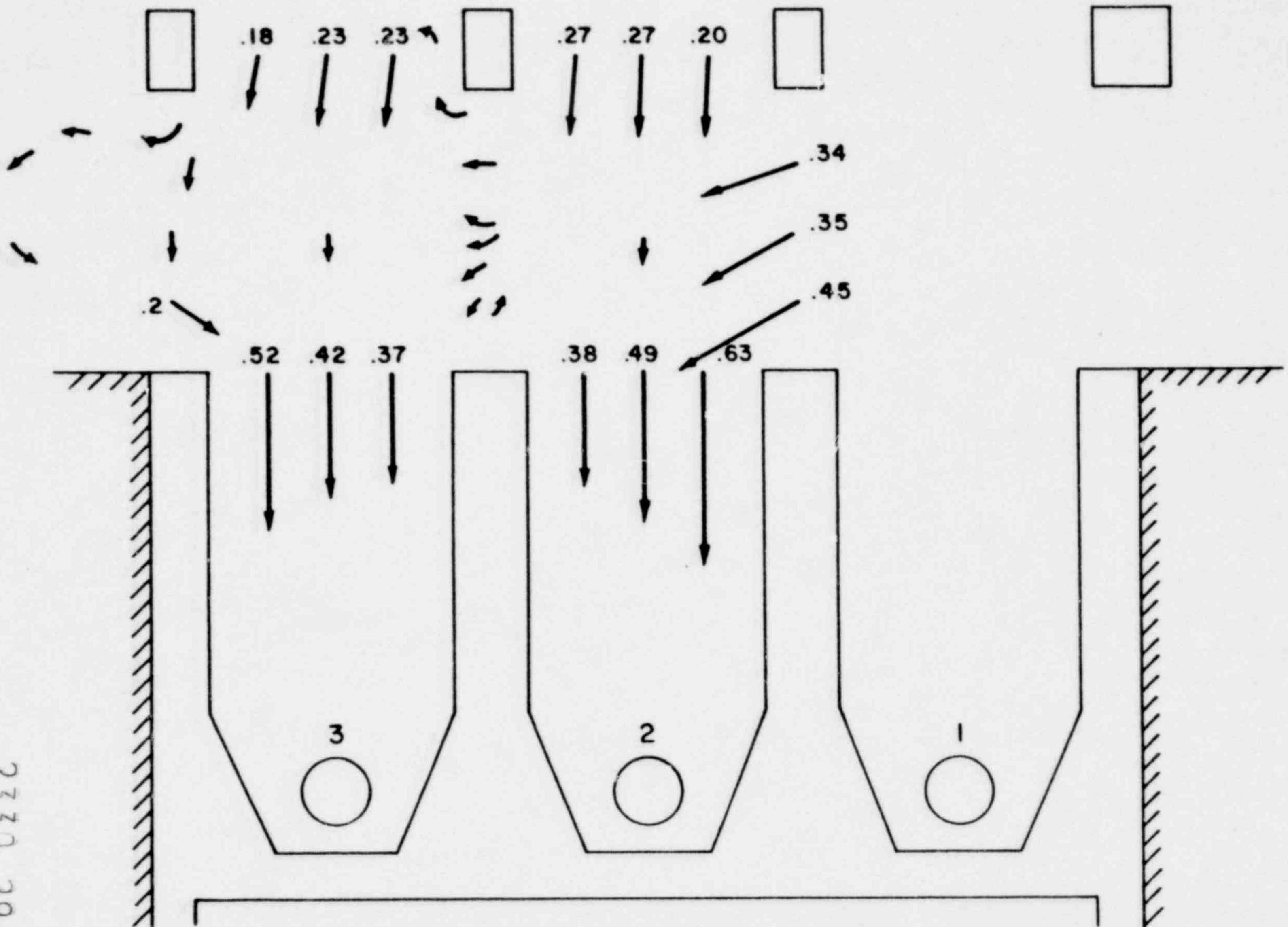

Figure 24. Flow pattern at intake structure no. 1 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 2 and 3 in operation.

2330 294

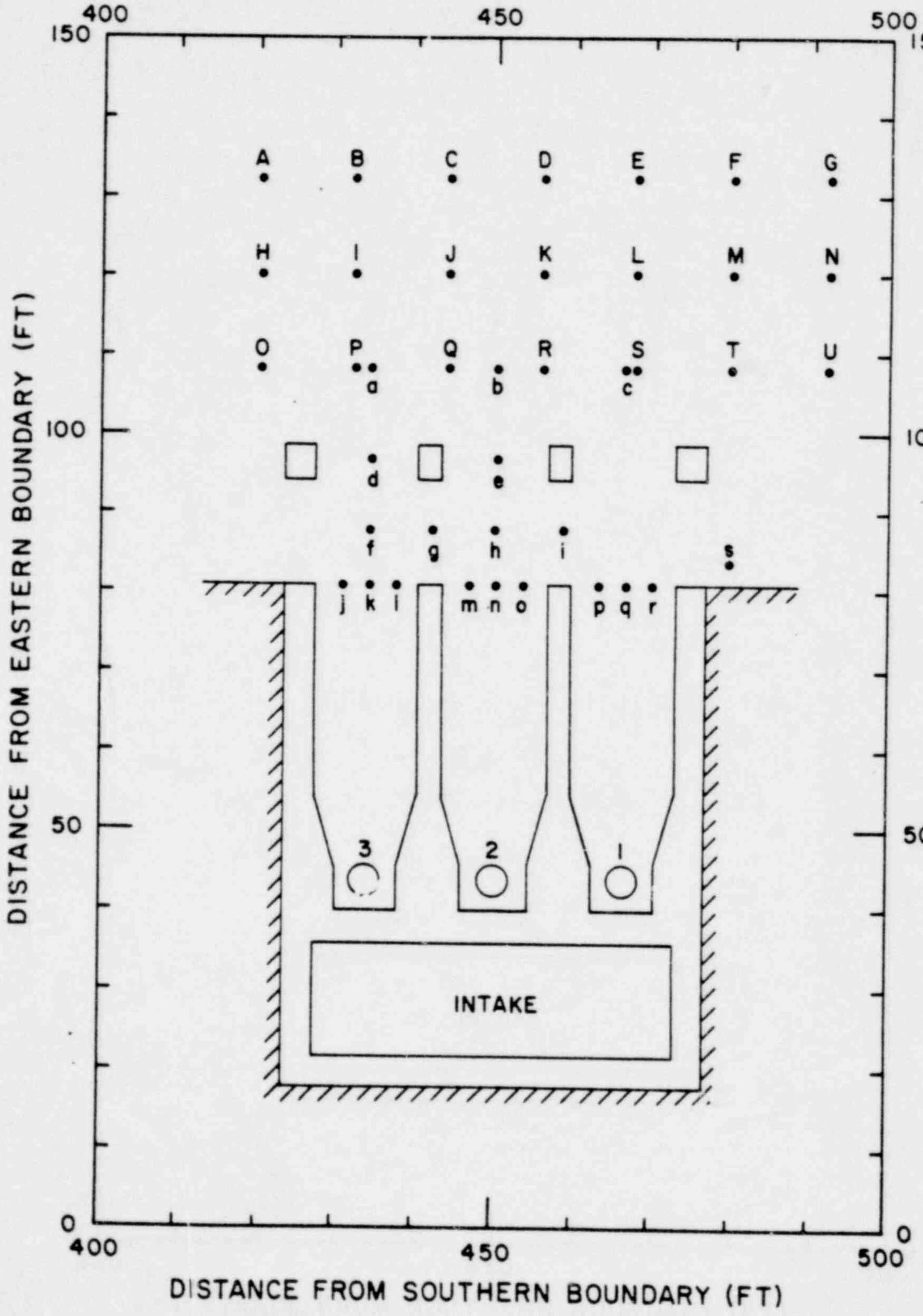

Figure 25. Points of measurement at intake structure no. 2

2330 295

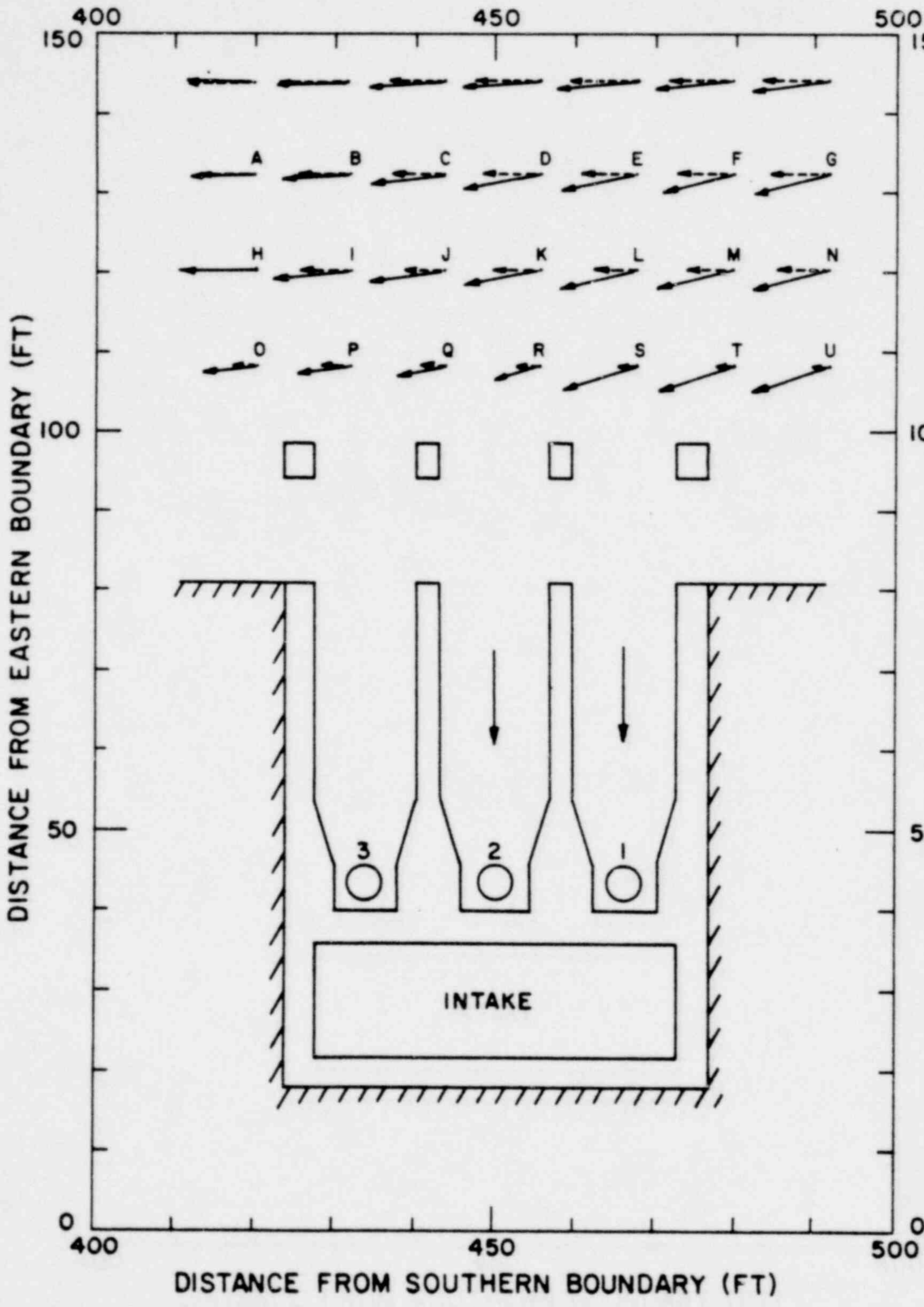

Figure 26. Flow pattern at intake structure no. 2 with and without withdrawal at the 7-day 10-year low flow for April and May; stage = 583 ft.

2330 296

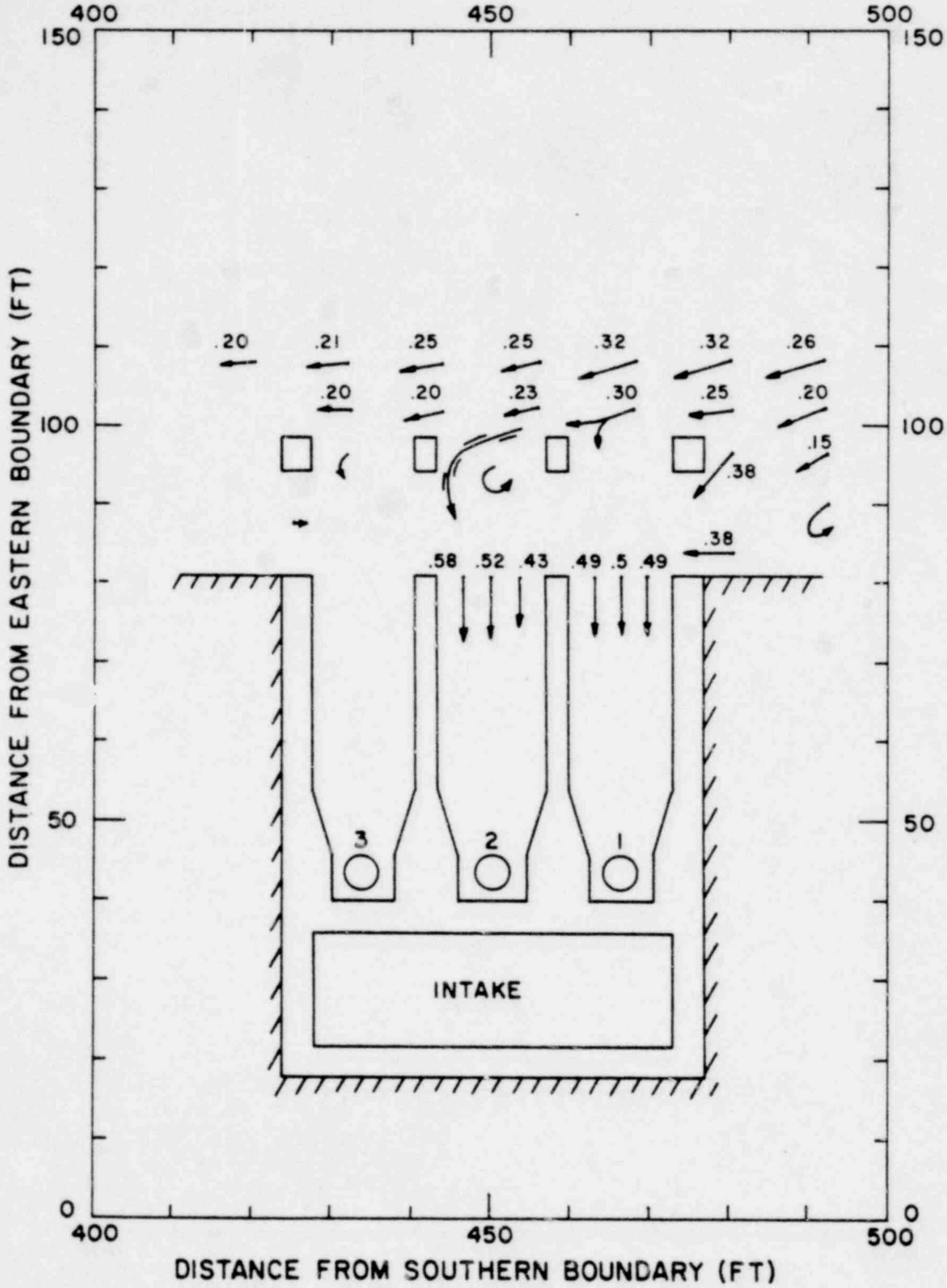

Figure 27. Flow pattern at intake structure no. 2 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 1 and 2 in operation

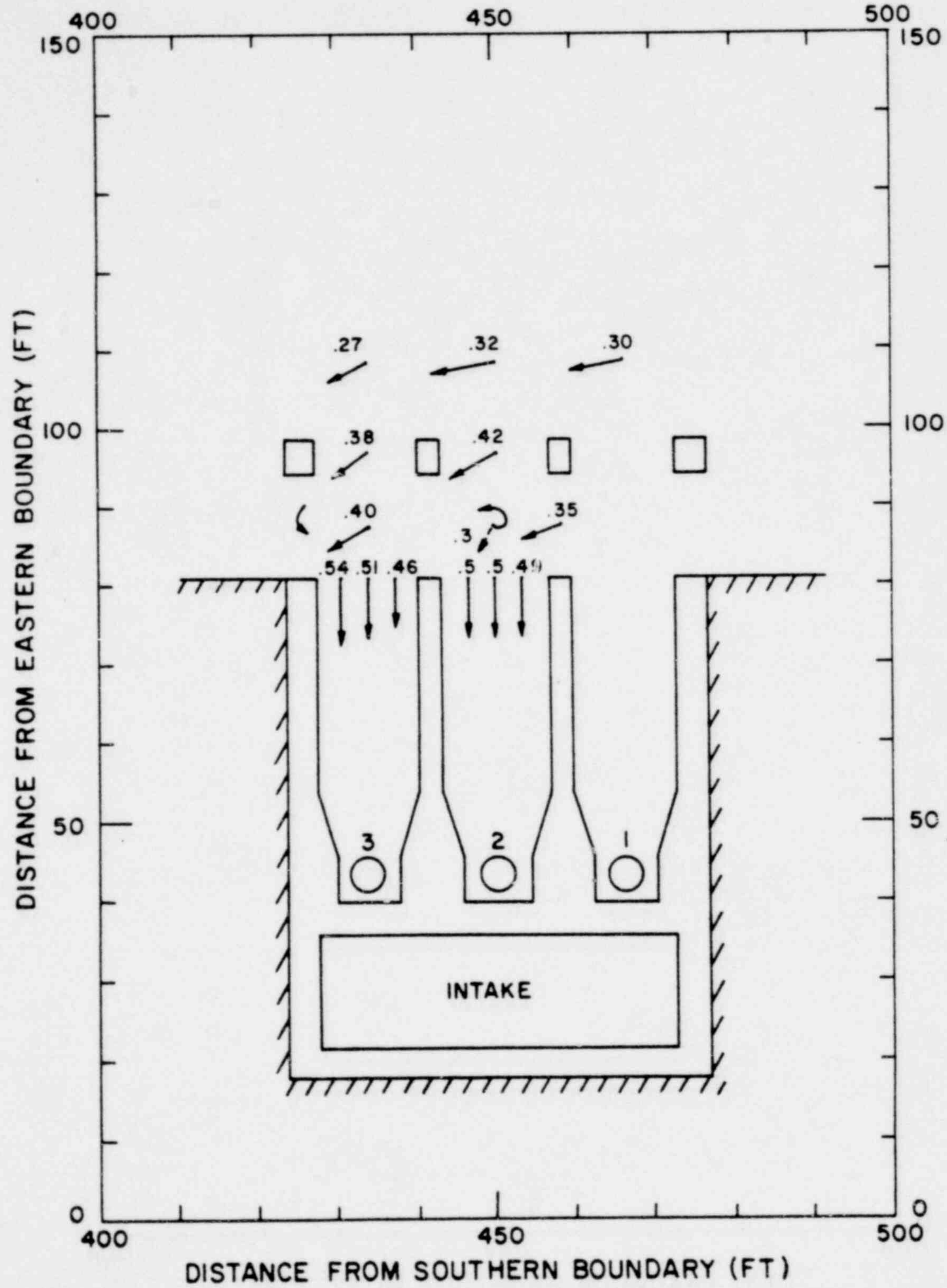

Figure 28. Flow pattern at intake structure no. 2 at the 7-day 10-year low flow for April and May; stage = 583 ft; pumps 2 and 3 in operation

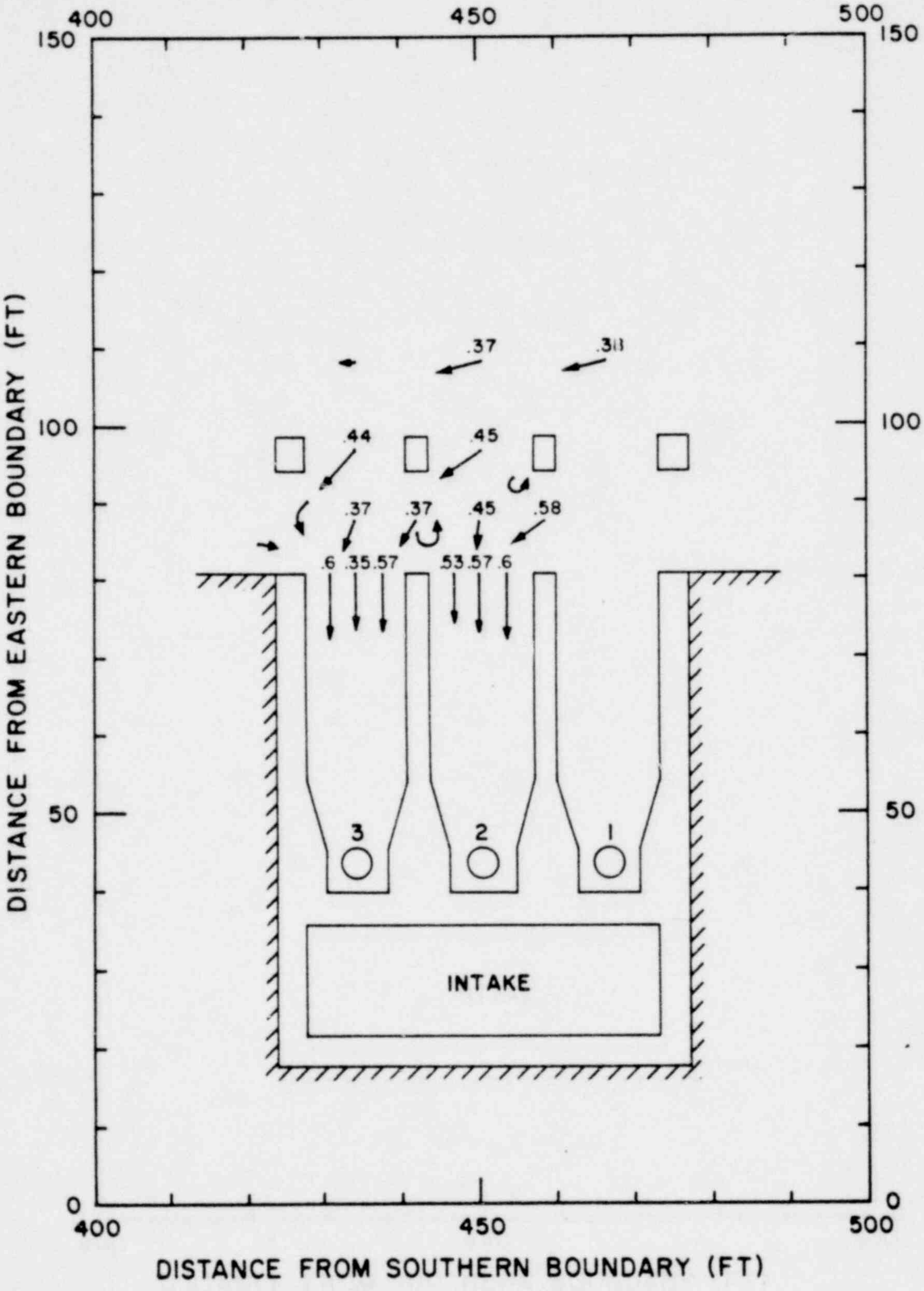

Figure 29. Flow pattern at intake no. 2 at the annually based 7-day 10-year low flow; stage = 582 ft; pumps 2 and 3 in operation

2330 299