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S Army Corps
f Engineers

ST. JOHNS BAYOU PUMPING STATION, MISSOURI

Hydraulic Model Investigation

by

B. P. Fletcher

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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13. ABSTRACT *(Maximum 200 words)*

The design of the proposed St. Johns Pumping Station at New Madrid, MO, consisted of three vertical pumps with a total capacity of 1,000 cfs.

A 1:11.5-scale pumping station model of the pump intakes, sump, and inlet channel was used to investigate and develop a practical design that would provide satisfactory hydraulic performance.

The model tests revealed that prototype construction costs could be reduced by reducing the sump length and the angle of the approach wing walls.

Initially, adverse flow distribution and excessive swirl were measured in the pump intakes. A pump intake design was developed that had insignificant swirl and provided good flow distribution for anticipated flow conditions.

14. SUBJECT TERMS

Pump intake	Swirl
Pump sump	Vortices
Pumping station	

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PREFACE

The study of the sump for the St. Johns Bayou Pumping Station, New Madrid, MO, was authorized by the Headquarters, US Army Corps of Engineers (HQUSACE), on 31 August 1987 at the request of the US Army Engineer District, Memphis.

The study was conducted during the period September 1987 to June 1989 in the Hydraulics Laboratory of the US Army Engineer Waterways Experiment Station (WES) under the direction of Messrs. F. A. Herrmann, Jr., Chief, Hydraulics Laboratory; R. A. Sager, Assistant Chief, Hydraulics Laboratory; and G. A. Pickering, Chief, Structures Division (SD), Hydraulics Laboratory. The tests were conducted by Messrs. G. R. Triplett, J. R. Rucker, and B. P. Fletcher, Spillways and Channels Branch (SCB), SD, under the direct supervision of Mr. N. R. Oswalt, Chief, SCB. This report was prepared by Mr. Fletcher and edited by Mrs. Marsha C. Gay, Information Technology Laboratory, WES.

During the course of the study, Messrs. Tom Munsey, HQUSACE; Larry Holman, Joe McCormick, and Larry Eckenrod of the US Army Engineer Division, Lower Mississippi Valley; and John Harman, Steve Barry, Harold Stricker, and David Berretta of the Memphis District visited WES to observe the model in operation, formulate plans for future tests, and correlate test results with concurrent design work.

Commander and Director of WES during preparation of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet of water (39.2° F)	2,988.98	pascals
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
square miles	2.589998	square kilometres

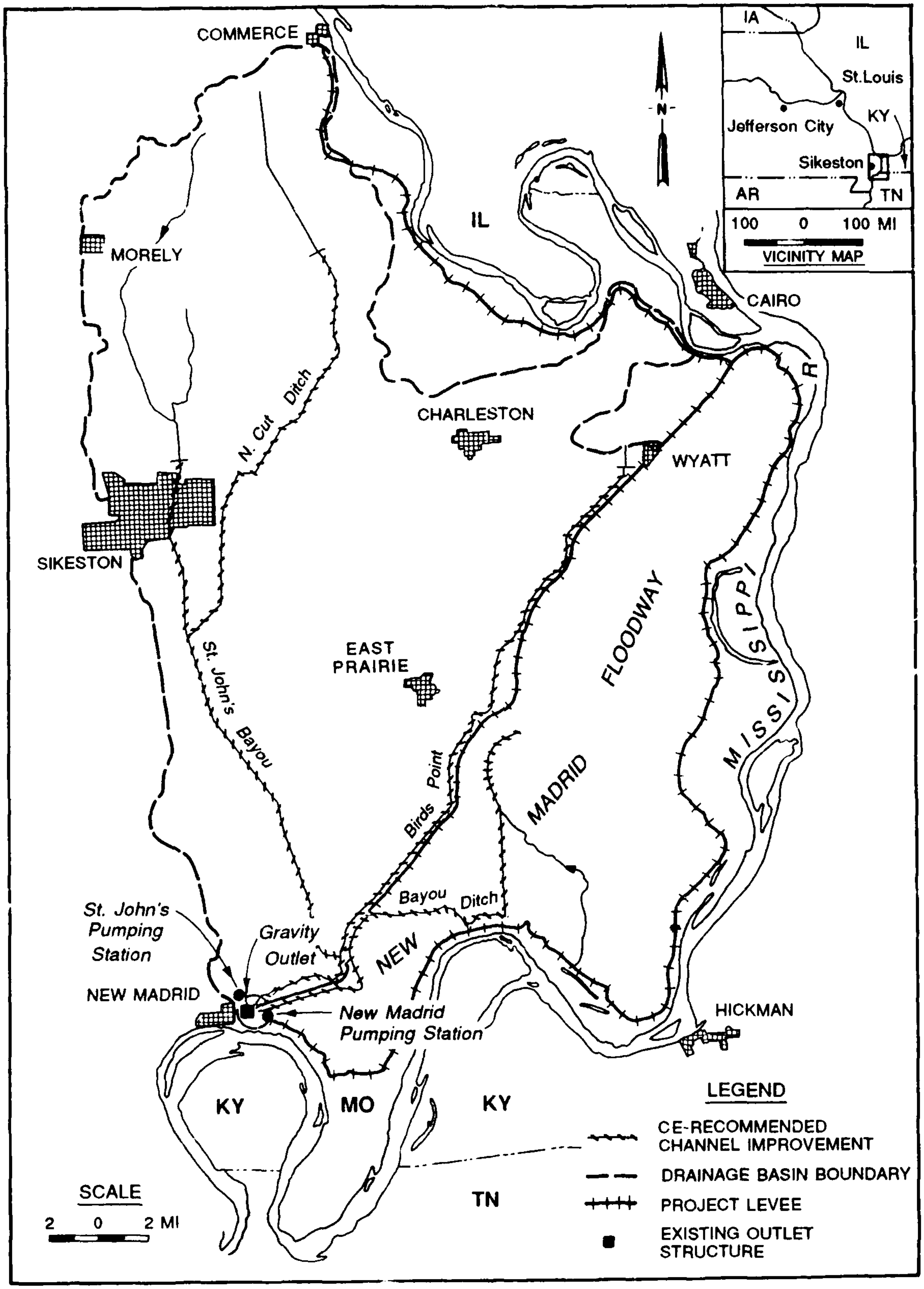

Figure 1. Location and vicinity map

ST. JOHNS BAYOU

PUMPING STATION, MISSOURI

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The St. Johns pumping station will be located in the St. Johns drainage basin, which is adjacent to the New Madrid Floodway, New Madrid, MO. The New Madrid Floodway also includes a proposed adjacent pumping station (New Madrid), a model of which was concurrently studied at the US Army Engineer Waterways Experiment Station (WES). Test results obtained from the model of the New Madrid pumping station are presented in a separate report.* The St. Johns Bayou and New Madrid Floodway basins are located in southeast Missouri (Figure 1) and include all or portions of New Madrid, Scott, and Mississippi Counties. The basins are adjacent to the Mississippi River, extending from the vicinity of Commerce, MO, to New Madrid. The area is divided by ridge lines and levees into the two distinct drainage basins. The relative proposed locations of the two pumping stations are shown in Figure 2. The St. Johns and New Madrid pumping stations would provide an outlet for floodwaters impounded during high stages on the Mississippi River.

2. St. Johns drainage basin is approximately 450 square miles** and is fan shaped with a length of about 40 miles and a maximum width of 25 miles. Runoff from St. Johns Basin drains through an existing gravity outlet (Figures 1 and 2) consisting of six 10- by 10-ft box culverts completed in 1953. During periods of high water on the Mississippi River, approximately 29 ft on the New Madrid gage, the floodgate structure will prevent Mississippi River backwater flooding in the St. Johns Bayou watershed. When the floodgates are closed, the flow is impounded until the Mississippi River recedes to

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* J. R. Leech. 1990 (Sep). "New Madrid Pumping Station, Gravity Flow Conduit and Confluence, New Madrid Floodway, Missouri; Hydraulic Model Investigation," Technical Report HL-90-12, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

** A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

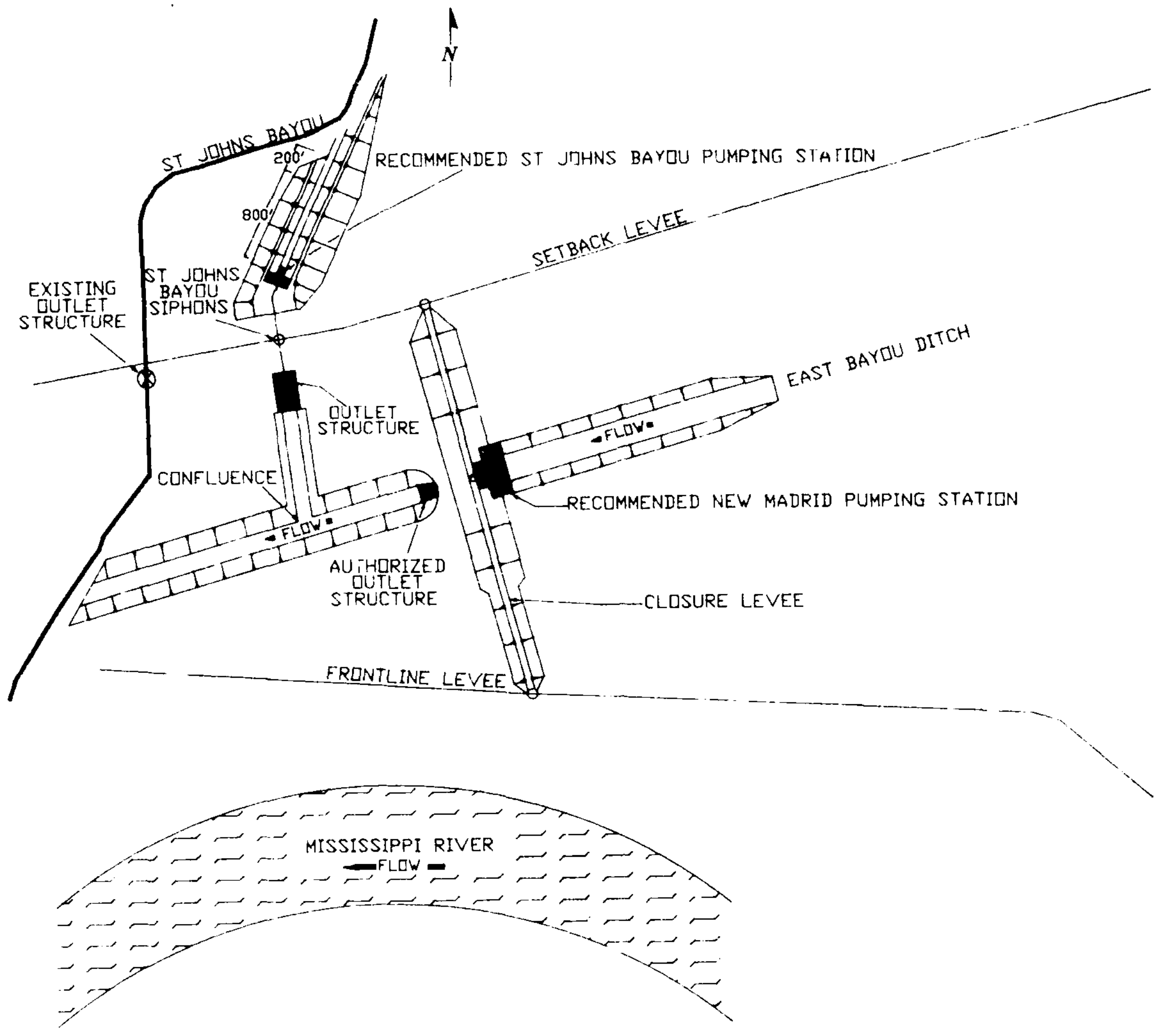

Figure 2. Locations of proposed pumping stations

an elevation lower than the impounded landside water.

3. The St. Johns pumping station recommended plan consists of an inlet channel, intake structure, pump house, discharge pipes, outlet structure, and outlet channel. The inlet channel will be improved for 800 ft landside of the intake structure and then have a 200-ft transition to the existing ditch (Figure 2). Bottom width of the improved channel will be approximately 65 ft (Figure 3). The ditch bottom grade will be el 269.5* and the side slopes will be 1V on 3.75H to el 282.0. The banks will have a 30-ft berm at el 282.0 and then the slope will continue up to a training dike with a crown el 290.0.

* All elevations (el) and stages cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Inlet channel wing walls will be used to retain fill at the pump house and to transition flow from the inlet channel to the intake structure.

4. Three pump bays will be located in the center of the pump house. The invert of the pump bay sump will be at el 269.0. A catenary trashrack will be located in the forebay. Gates will be located in the forebay to seal the bays during nonpumping periods. Each pump bay will employ a vertical propeller-type pump having a formed suction intake. The pumps will pump water over the levee through 72-in. discharge pipes using siphonic recovery. To seal the end of the pipe and initiate prime during low river stages, a saxophone discharge arrangement will be used. To limit the vacuum in the top of the discharge pipes to 28 ft of water, the lip of the saxophone outlet will be set at el 284.0. A separate physical model study was conducted to evaluate the hydraulic characteristics of the siphon. Results of this study have been documented in a separate report.* Each pump will be capable of pumping a design flow of 333.3 ft^3/sec at the average static head of 3.0 ft. The average static head condition of 3.0 ft is the difference between average river stage (el 290.1) and average sump water surface (el 287.1). The storm water pumps will be manually started when the sump elevation reaches 279.0 and automatically stopped when the water elevation in the sump drops to 277.0.

Purpose and Scope of the Model Study

5. The model study was conducted to evaluate the hydraulic characteristics of and develop modifications required for a satisfactory design of the approach channel, sump, and formed suction intake. Tests were conducted for the range of anticipated discharges and water-surface elevations and for various combinations of pumps operating.

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* D. R. Cooper. "St. Johns Bayou Pumping Station Siphon" (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, MS.

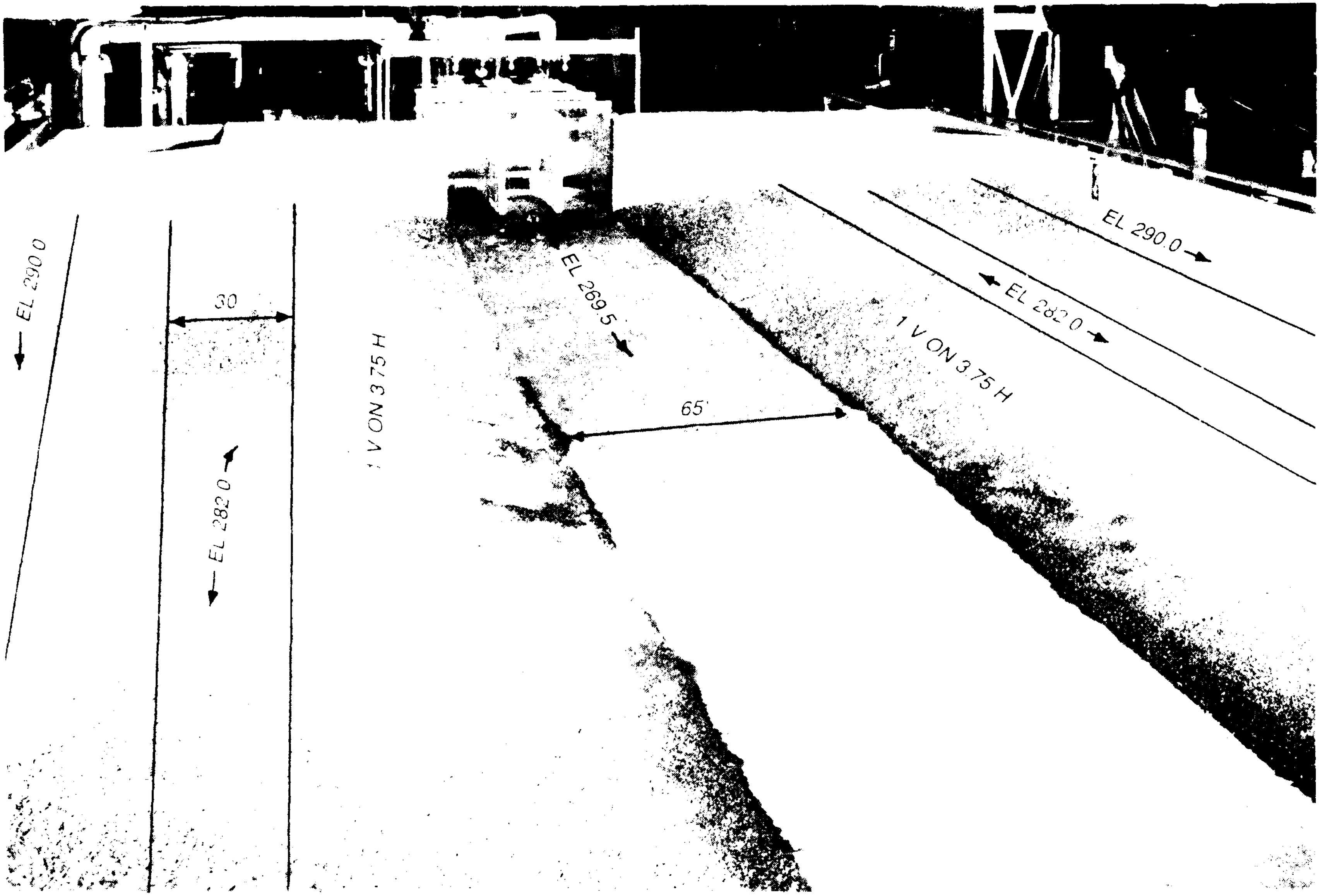


Figure 3. The 1:11.5-scale model

PART II: THE MODEL

Description

6. The 1:11.5-scale model (Figure 3) reproduced a 425-ft length and 333-ft width of approach to the sump, the sump, three pump bays, and pump intakes. The geometry of the approach channel was simulated by pea gravel (Figure 3) to facilitate modifications to the approach channel geometry. The sides, interior walls, and pump intakes were constructed of transparent plastic to permit observation of vortices, turbulence, and subsurface currents. Flow through each pump intake was provided by individual suction pumps that permitted simulation of various flow rates through one or more pump intakes.

7. Water used in the model was recycled and discharges were measured with turbine flowmeters.

Scale Relations

8. The model was sized so that the Reynolds number, defined as

$$R = \frac{Vd}{\gamma} \tag{1}$$

where

V = average velocity, ft/sec

d = diameter of pump suction column, ft

γ = kinematic viscosity of fluid, ft^2/sec

is greater than 10^5 to minimize scale effects due to viscous forces.

9. The accepted equations of hydraulic similitude, based upon Froudian criteria, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and prototype. The general relations expressed in terms of the model scale or length ratio L_r are presented in the following tabulation. Measurements of discharge, water-surface elevation, head, velocity, and time can be transferred quantitatively from the model to prototype equivalents by means of the scale relations.

Dimension*	Ratio	Scale Relations Model:Prototype
Length	L_r	1:11.50
Area	$A_r = L_r^2$	1:132.25
Velocity	$V_r = L_r^{1/2}$	1:3.39
Discharge	$Q_r = L_r^{5/2}$	1:448.48
Time	$T_r = L_r^{1/2}$	1:3.39
Pressure	$P_r = L_r$	1:11.50

* Dimensions are in terms of length.

PART III: TESTS AND RESULTS

Evaluation Techniques

10. Techniques used for evaluation of hydraulic performance include the following:

a. Current patterns in the approach channel were determined using dye injected into the water and confetti sprinkled on the water surface. Water-surface elevations were measured with staff and point gages. Velocities in the approach channel and pump bays were measured with pitot tubes and electromagnetic velocity probes.

b. Visual observations were made to detect surface and/or submerged vortices. A design that permits a Stage C surface vortex or submerged vortex with a visible air core is considered unacceptable. Stages of surface vortex development are shown in Figure 4. A typical test consisted of documenting, for a given flow condition, the most severe vortex that occurred in a 5-min (model time) time period.

c. Swirl angle was measured to indicate the strength of swirl entering the pump intake. A swirl angle that exceeds 3 deg is considered unacceptable. Swirl in the pump columns was indicated by a vortimeter (free-wheeling propeller with zero-pitch blades) located inside the pump column (Plate 1). Swirl angle is defined as the ratio of the blade speed at the tip of the vortimeter blade V_θ to the average velocity V_a for the cross section of the pump column. The swirl angle θ is computed from the following formula:

$$\theta = \tan^{-1} \frac{V_\theta}{V_a} \quad , \quad V_\theta = \pi dn \quad , \quad V_a = \frac{Q}{A} \qquad (2)$$

where

θ = swirl angle, deg

V_θ = tangential velocity at the tip of the vortimeter blade, ft/sec

V_a = average pump column axial velocity, ft/sec

d = pump column diameter (used for blade length), ft

n = revolutions per second of the vortimeter

Q = pump discharge, ft^3/sec

A = cross-sectional area of the pump column, ft^2

d. Velocity distribution and flow stability in the pump intakes were measured by impact tubes located in the pump columns (Figures 5 and 6). Cross sections at the tips of the impact tubes (el 277.0) are shown in Plates 2 and 3. A deviation in

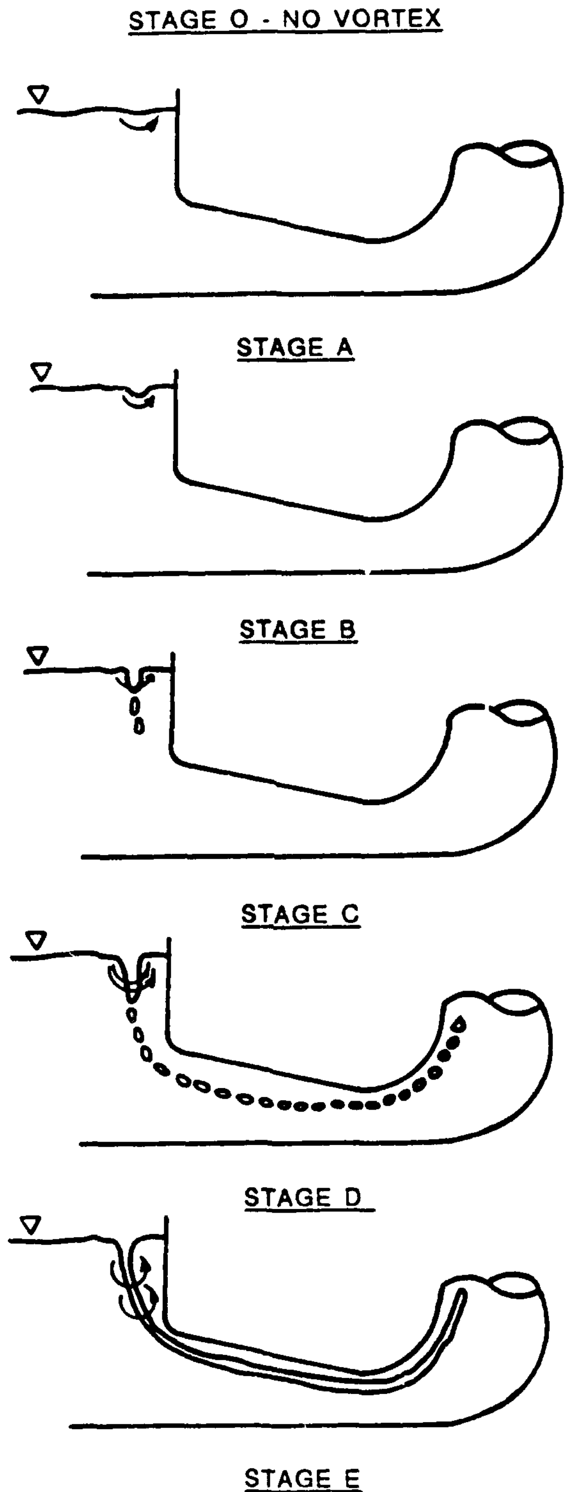


Figure 4. Stages in surface vortex development, formed suction inlet

Figure 5. Type 1 pump intake

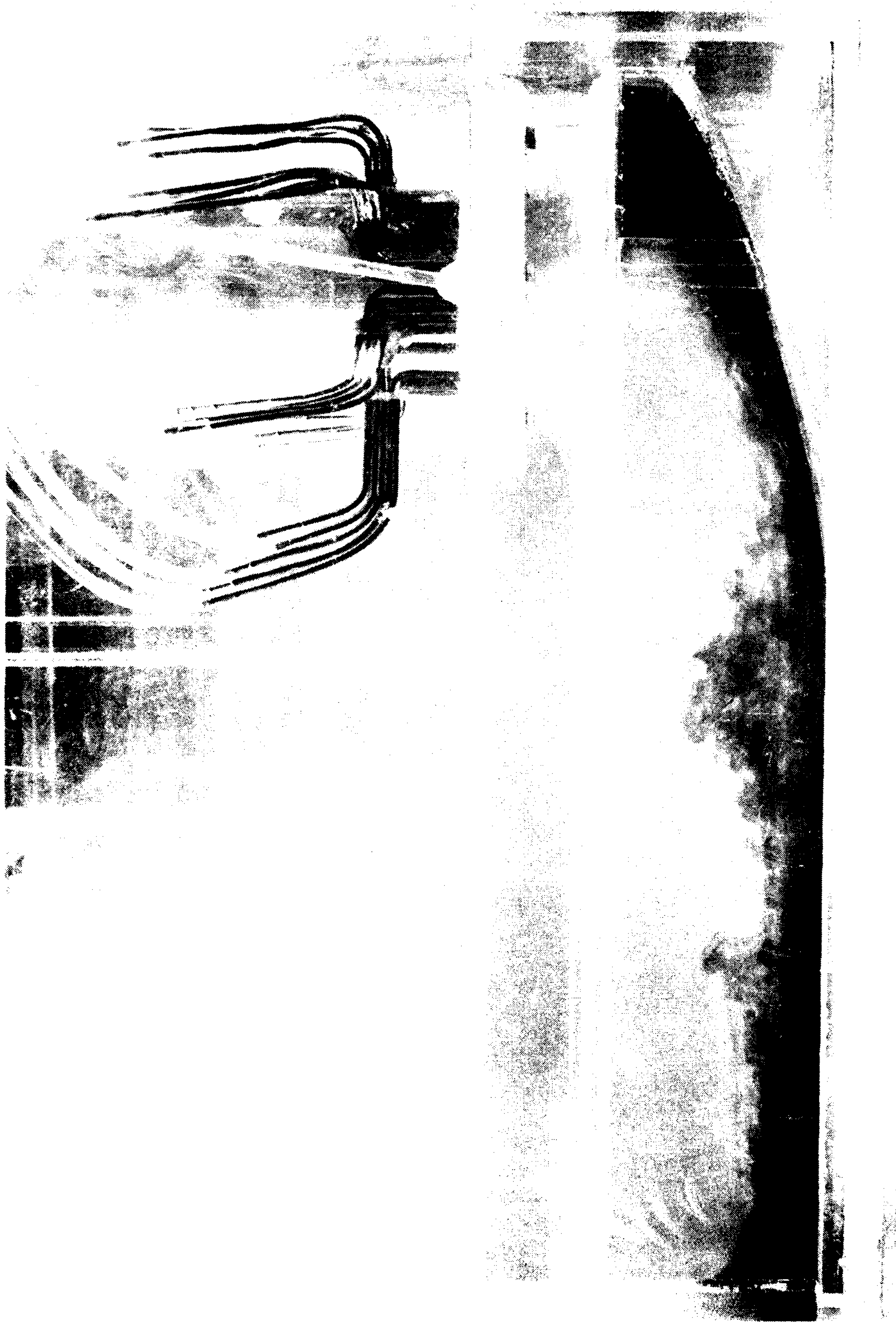


Figure 6. Type 2 pump intake

the ratio of the average measured velocity at a point to the average computed velocity in the cross section of 10 percent or greater was considered unacceptable. Four piezometers were located around the periphery of the pump column (Plates 2 and 3) to measure an average static pressure at this location. Impact tubes (copper tubes with 1/8-in. ID) were installed with their tips in the same plane as the four piezometers to measure the total pressure at 25 various points (Plates 2 and 3) in the pump column. The head differential between the total pressure at each point in the pump column and the average static pressure can be used to determine a velocity at each point in the pump column. This was measured using 25 individual electronic pressure differential cells (Figure 7). The differential cells were connected to a data acquisition system capable of collecting data for various lengths of time and sampling at various rates. The data acquisition system was also capable of analyzing the data and providing the deviation in velocity ratio for each probe in the same timeframe that the maximum instantaneous velocity ratio deviation for any single probe occurred. The magnitude of the maximum velocity deviation that should be considered unacceptable has not been established.

11. A typical test to measure velocity distribution in the pump column consisted of stabilizing the water-surface elevation and discharge through the pump prior to collecting data. Data were collected for 1 min (model time) and each of the 25 differential pressure cells was sampled at a rate of 100 samples per second. The minimum, average, and maximum velocities detected by each of the differential cells during the minute of data collection were divided by the theoretical average velocity in the cross section. The ratio (measured/computed) of the average velocities and ratio (measured/computed) of the velocities at all points that occurred in the same timeframe of the maximum velocity deviation ratio anywhere in the cross section were tabulated and plotted by a computer as contour lines of equal velocity ratios. The ratio of the average velocities and the ratio of the velocities that occurred in the same timeframe of the maximum velocity deviation were used as parameters for evaluating flow conditions, because the average velocity was an indicator of flow distribution and the maximum velocity ratio deviation was sensitive to a change in flow stability.

Inlet Channel and Sump

12. A sketch of the type 1 sump design is shown in Plate 4. After analysis of the design and discussions with personnel from the US Army

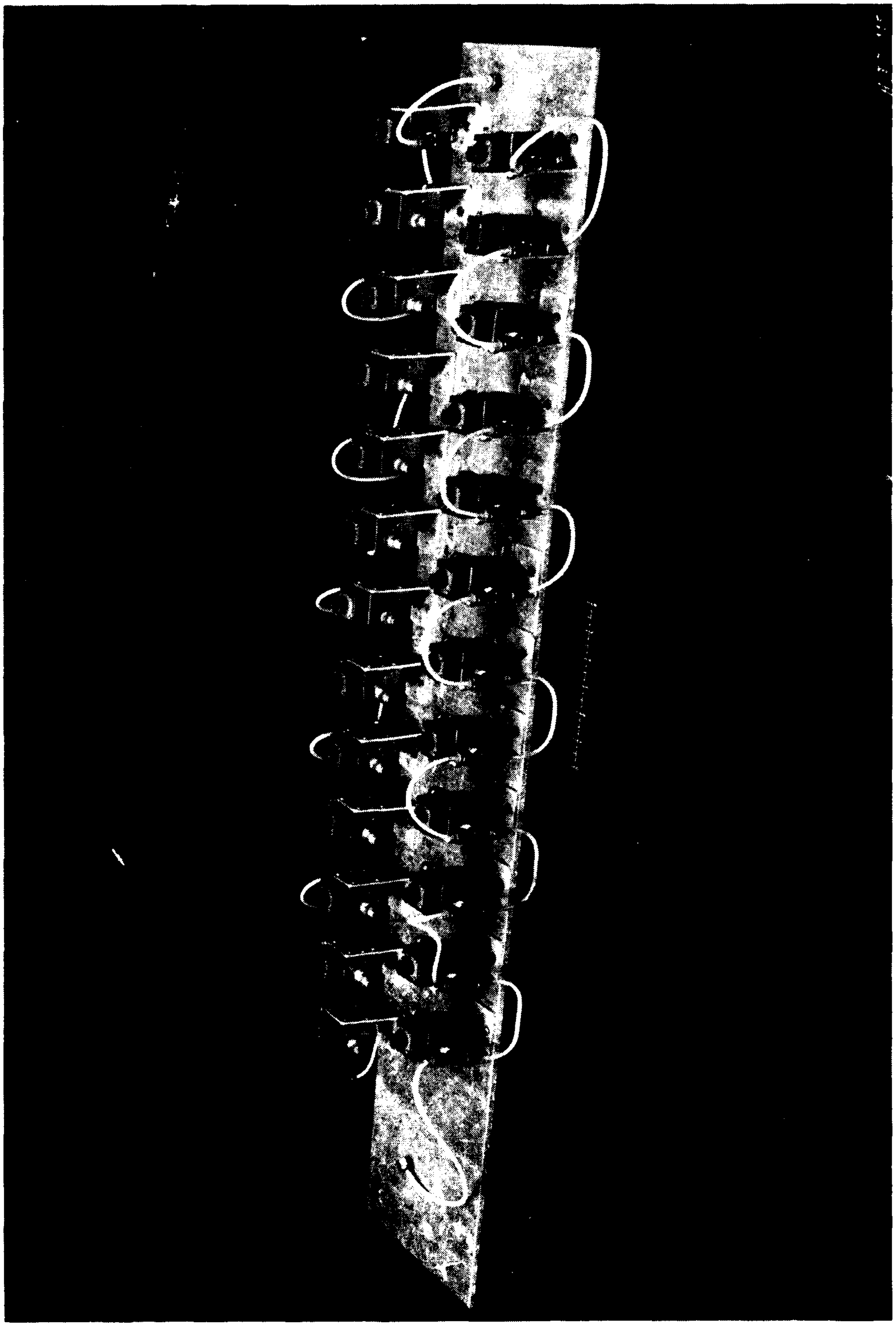


Figure 7. Pressure cells

Engineer District, Memphis, it was decided to remove the U-section shown in Plate 4. Reducing the sump length by removal of the U-section would reduce prototype construction costs. Initial tests conducted with the U-section removed (type 2 sump, Plate 5) revealed that the U-section was not needed to provide satisfactory approach flow in the sump. A sketch of the type 1 inlet channel and the type 2 sump is shown in Plate 6.

13. In the interest of economy, the wing wall angle was changed from 45 to 30 deg (type 3 sump, Plates 7 and 8). Hydraulic performance was satisfactory and similar to that observed with the type 2 sump. Surface currents for the type 1 inlet channel and type 3 sump design generated by various combinations of pumps operating at the minimum anticipated water-surface elevation and the maximum discharge are depicted in time-exposed photographs of confetti (Photo 1). Flow patterns and bottom velocities are shown in Plate 9. The model tests indicated that flow in the type 1 inlet channel and type 3 sump was stable, well distributed, and satisfactory for all anticipated flow conditions.

Pump Intakes

Type 1 and 2 designs

14. The three pump intakes shown in Plate 8 are identified from left to right, facing downstream, as pumps 1, 2, and 3. Two pump intake designs were simultaneously simulated in the model to provide a comparison in hydraulic performance. The type 1 pump intake design (Plate 1 and Figure 5) was simulated in pump bay 1. The type 2 pump intake design (Plate 10 and Figure 6) was simulated in pump bays 2 and 3. The geometry of the approach channel and sump was symmetrical, and flow patterns in pump bays 1 and 3 were similar for respective flow conditions.

15. Indicators describing hydraulic performance in the type 1 and 2 pump intake designs with the type 1 inlet channel and type 3 sump are listed in Table 1. Flow in the type 1 and 2 pump intakes produced no significant surface or submerged vortices. A comparison of the swirl angles (Plate 11) obtained from the vortimeter readings (Table 1) indicates that the type 2 pump intake design was subject to excessive swirl (swirl angle greater than 3 deg) and the type 1 intake provided satisfactory hydraulic performance with swirl angles less than 3 deg.

16. The type 1 and 2 pump intake designs were further evaluated by measuring velocity distribution at the approximate location of the pump propeller. Velocity distribution in the type 1 pump intake design (Plate 1) was documented with the minimum anticipated water-surface el of 277.0 and the maximum anticipated discharge of 333 ft^3/sec. The ratios of the measured average velocity to the computed average velocity in the pump column with one and three pumps operating are shown as contour lines in Plates 12 and 13, respectively. An undesirable zone of low velocity occurred immediately downstream from the roof curve and is depicted by the contour lines in the upper quadrant in Plates 12 and 13. The plots of maximum velocity ratio deviation are shown in Plates 14 and 15.

17. The type 2 pump intake (Plate 10) was investigated with hydraulic conditions identical to those evaluated with the type 1 pump intake design. The ratios of the measured average velocity to the computed average velocity in the pump column with pump 3 operating are shown as contour lines in Plate 16 and with pumps 1, 2, and 3 operating in Plate 17. A zone of low pressure was observed in the upper quadrant with either one or three pumps operating. Plots of maximum velocity ratio deviation are shown in Plates 18 and 19.

18. A comparison of average velocity ratios in the type 1 and 2 pump intakes indicates that average flow distribution is better in the type 2 pump intake. However, the deviation in the velocity ratios in both pump intakes exceeds the acceptable value of 10 percent. The measured swirl angles were also greater in the type 2 pump intake.

Recommended design

19. The recommended design (type 3 pump intake) was developed from the type 1 pump intake design by reducing the pump column diameter from 6.0 to 5.2 ft and adding an 11-deg cone as shown in Plate 20. The cone provided streamlining that eliminated the zone of low pressure detected in the upper quadrant in the type 1 and 2 pump intake designs. The type 3 pump intake was installed in pump bay 1 (Plate 8).

20. Initially, flow distribution in the type 3 pump intake was investigated with the most severe anticipated hydraulic conditions: only pump 1 operating, minimum water-surface el 277.0, and the maximum discharge of 333.0 ft^3/sec. The ratios of the measured average velocity to the computed average velocity in the pump column with pump 1 operating are shown as contour

lines in Plate 21. The contour lines in Plate 21 indicate satisfactory flow distribution by not deviating more than 10 percent. A plot of maximum velocity ratio deviation is shown in Plate 22. Satisfactory performance was also indicated by measured swirl angles of less than 1.0 deg and no significant surface vortices (Table 1).

21. Additional observations and measurements during various submergences, discharges, and combinations of pumps operating confirmed that the type 1 inlet channel, type 3 sump, and type 3 pump intake (Plates 8 and 20) should provide satisfactory hydraulic performance for all anticipated flow conditions.

PART IV: DISCUSSION

22. Initially, analysis of the design and model tests confirmed that the length of the sump, and thus construction costs, could be reduced by removal of the U-section in the pump sump. Changing the approach wing wall angle from 45 to 30 deg did not adversely affect hydraulic performance and should also reduce prototype construction costs.

23. Two pump intake designs (types 1 and 2, Plates 1 and 10) were simultaneously simulated in the model to provide a comparison in hydraulic performance. The type 2 pump intake had excessive swirl (swirl angle 3 deg or greater). Swirl angles measured in the type 1 pump intake were acceptable, less than 3 deg.

24. Further evaluation was conducted by measuring flow distribution in the type 1 and 2 pump intakes. Both intakes experienced zones of low pressure that exceeded the acceptable limit as indicated by a greater than 10 percent deviation in cross-sectional velocity at the location of the impeller. Neither the type 1 nor 2 pump intake design met the design criteria for both swirl angle and flow distribution.

25. The type 3 pump intake design (recommended design) was obtained by adding an 11-deg cone to the type 1 pump intake design as shown in Plate 20. The type 3 pump intake design provided satisfactory performance while subjected to the most severe anticipated hydraulic conditions. Measurements and observations during various submergences, discharges, and combinations of pumps operating indicated that the type 1 inlet channel, type 3 sump, and type 3 pump intake should provide satisfactory hydraulic performance for all anticipated flow conditions.

Table 1

Hydraulic Performance

Type 1 Inlet Channel

Type 3 Sump

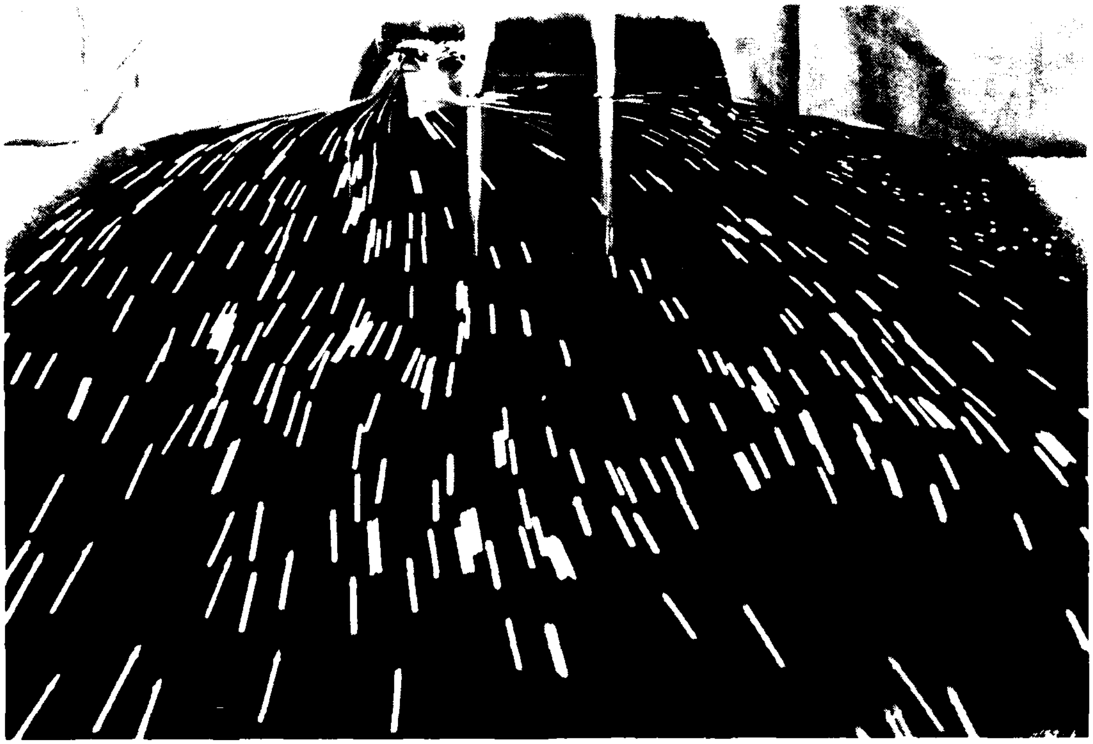
Type Pump Intake	Pump No.	Pumps Operating*	Water Surface El	Angle of Swirl deg**	Vortex Stage†
1	1	1	288.0	1.2→	0
	1	1, 2		1.3→	0
	1	1, 3		0.9→	0
	1	1, 2, & 3		1.3→	0
2	2	2		4.2→	A
	3	3		2.3→	0
	2	1, 2		3.7→	
	2	2, 3		4.8→	
	3	2, 3		2.4→	
	3	1, 3		2.1→	
	2	1, 2, & 3		3.5→	
	3	1, 2, & 3		2.0→	
1	1	1	277.0	0.9→	A
	1	1, 2		0.5→	0
	1	1, 3		0.9→	
	1	1, 2, & 3		0.7→	
2	2	2		3.9→	
	3	3		2.1→	
	2	1, 2		4.2→	
	2	2, 3		4.0→	
	3	2, 3		2.4→	
	3	1, 3		2.3→	
	2	1, 2, & 3		3.7→	
	3	1, 2, & 3		2.1→	
3	1	1		0.7→	A
	1	1, 2		0.5→	0
	1	1, 3		0.6→	
	1	1, 2, & 3		0.5→	
3	1	1	288.0	0.9→	
	1	1, 2	288.0	0.9→	
	1	1, 2, & 3	288.0	0.6→	
	1	1, 2, & 3	288.0	0.8→	

Note: All magnitudes are expressed in prototype equivalents.

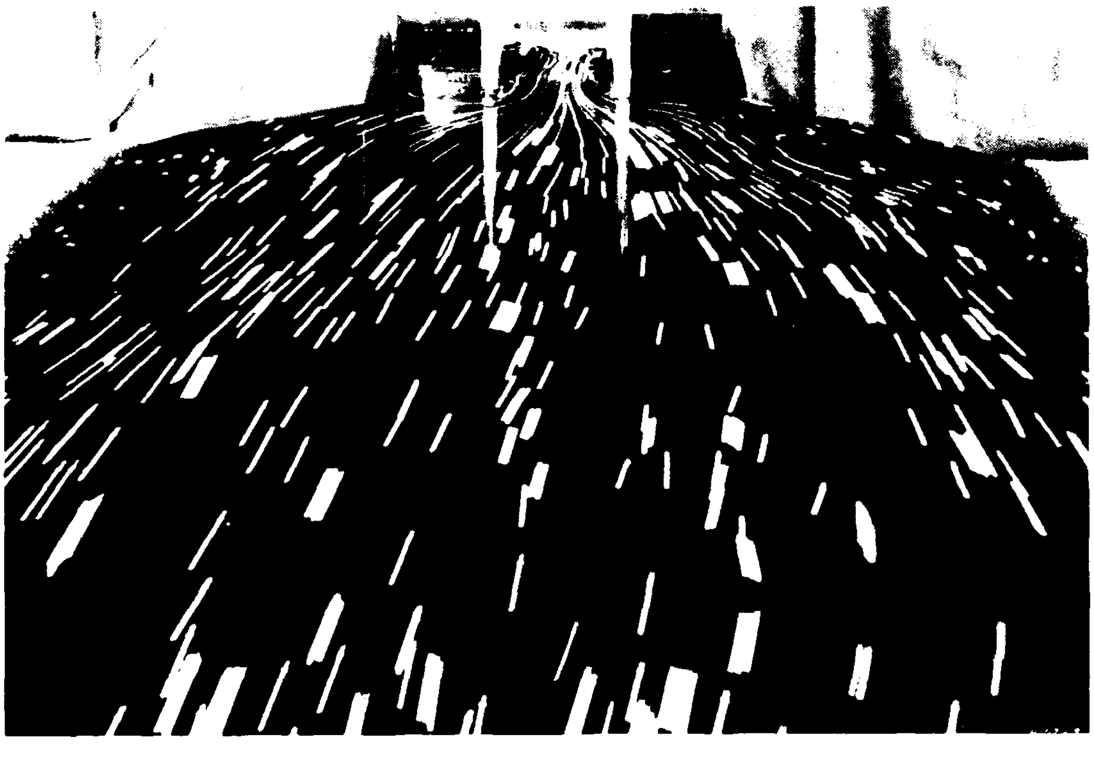
* Discharge per pump 333 ft^3/sec.

** → indicates clockwise rotation.

† See Figure 4.

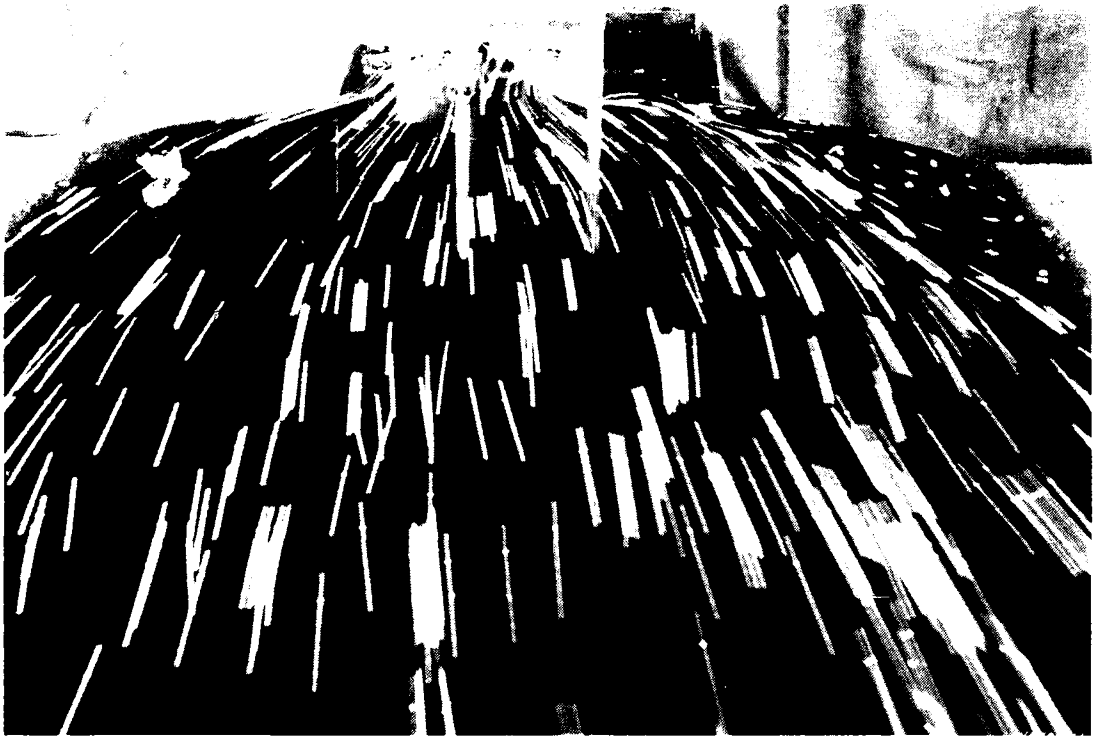


a. Pump 1 operating

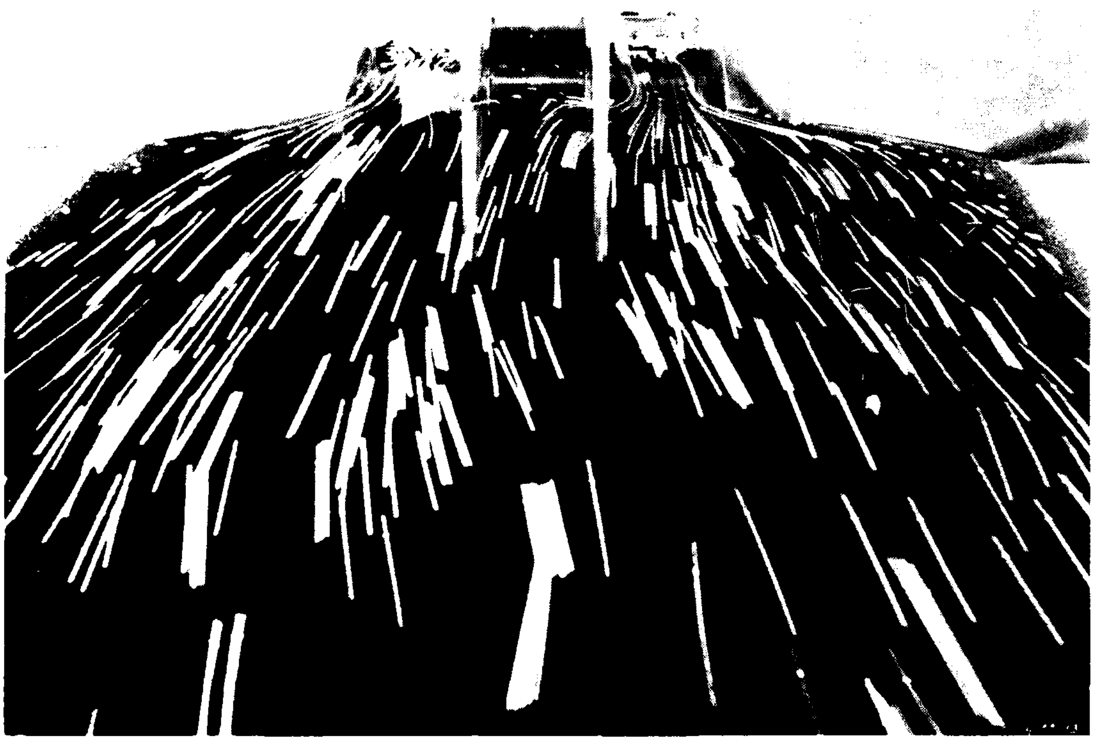


b. Pump 2 operating

Photo 1. Surface currents; type 1 inlet channel; type 3 sump; discharge 333 ft^3/sec per pump; water-surface el 277.0; exposure time 15 sec (prototype) (Sheet 1 of 3)

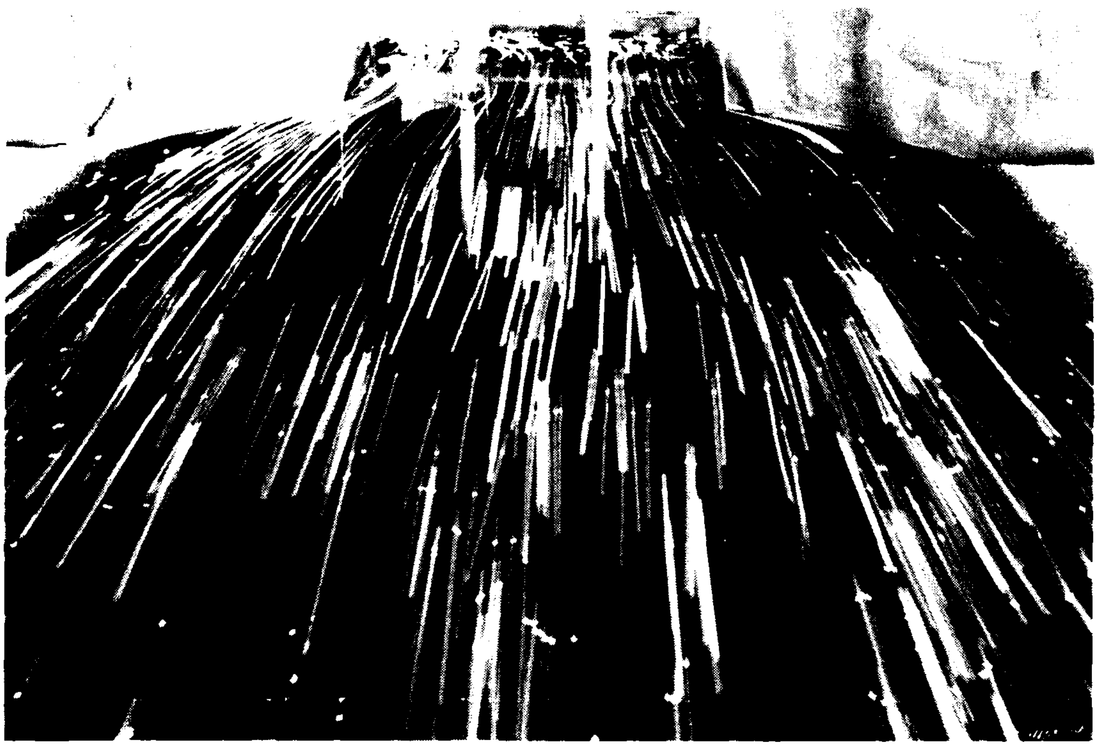


c. Pumps 1 and 2 operating



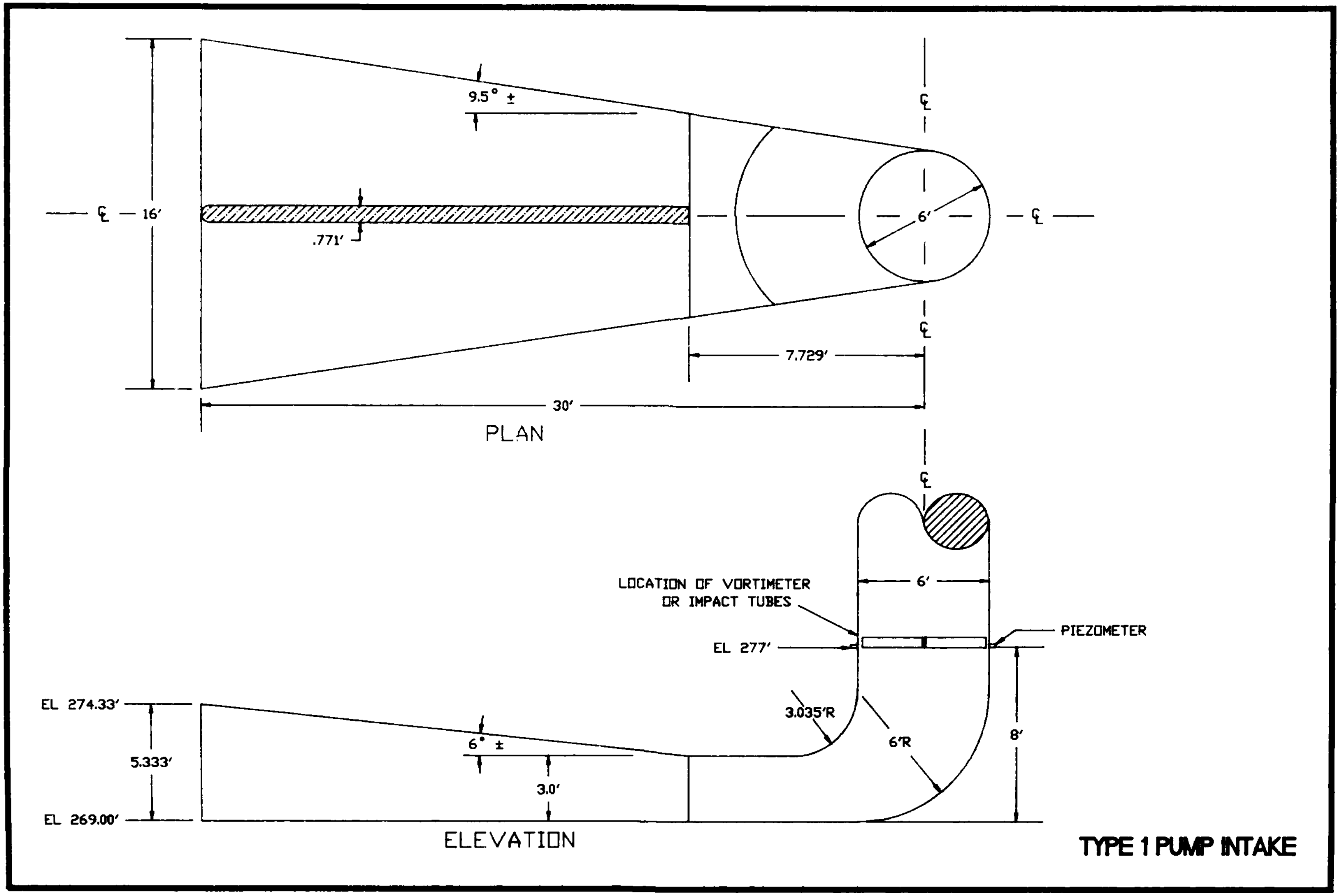
d. Pumps 1 and 3 operating

Photo 1. (Sheet 2 of 3)

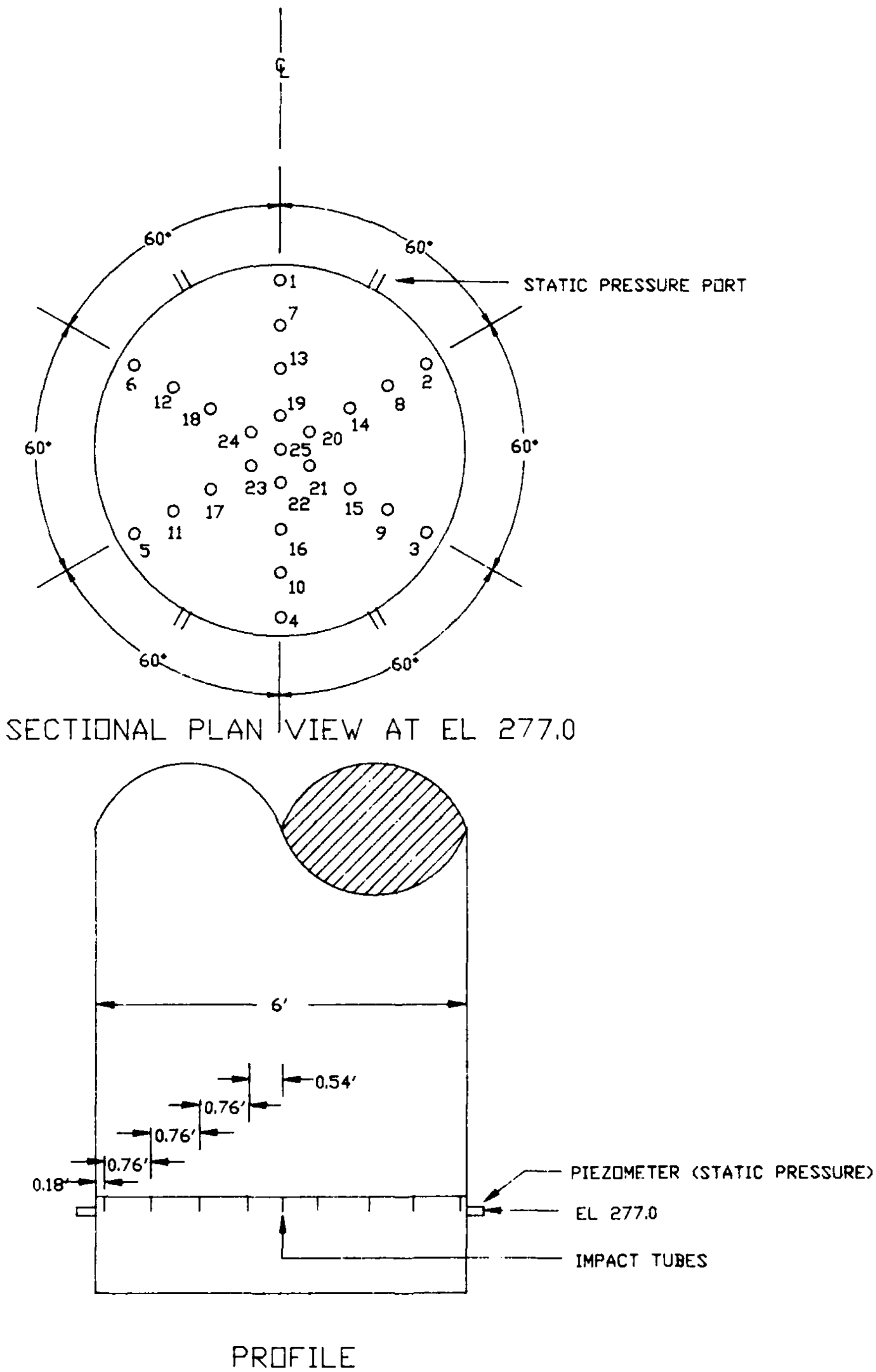


e. Pumps 1, 2, and 3 operating

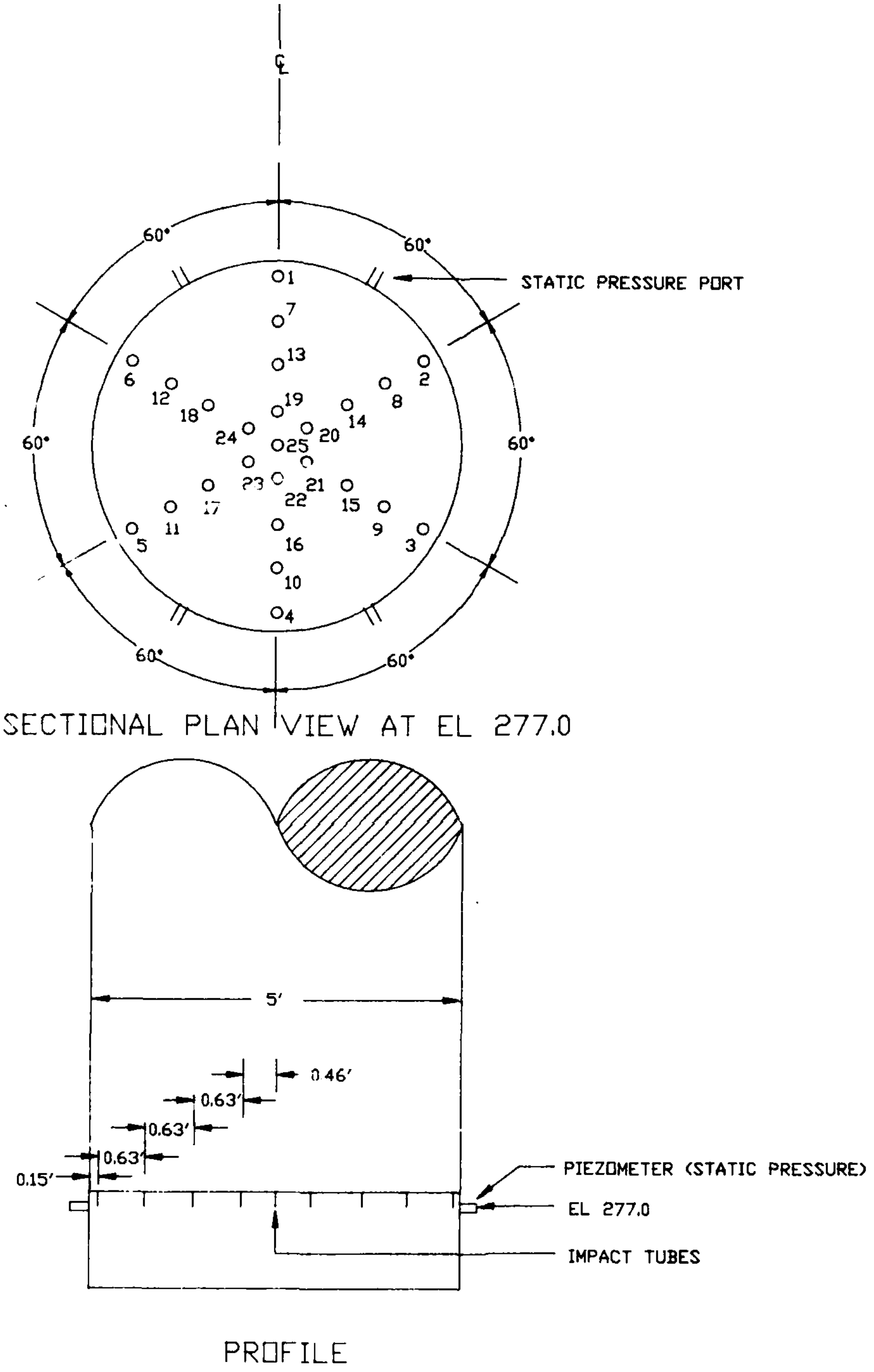
Photo 1. (Sheet 3 of 3)



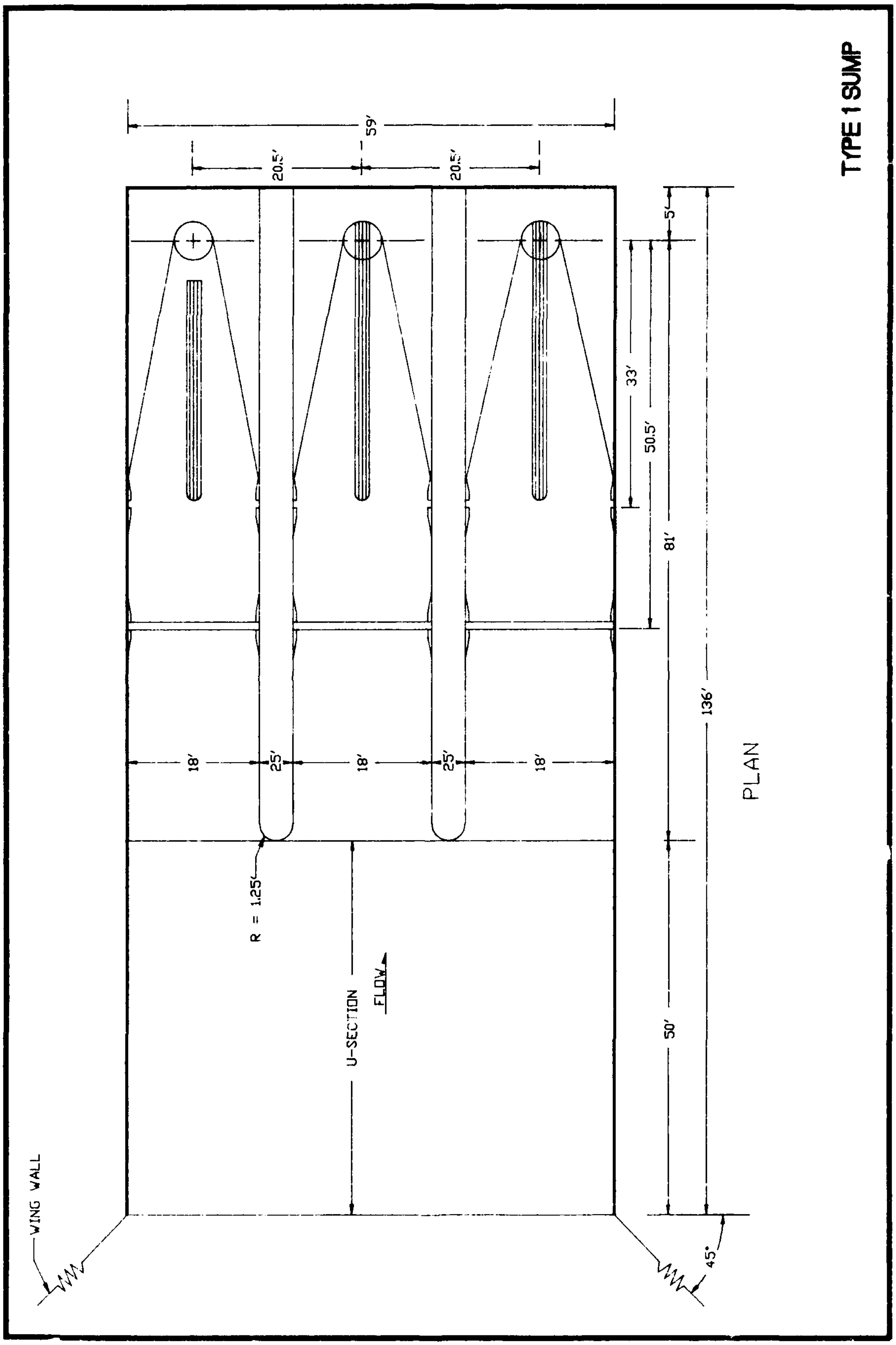
TYPE 1 PUMP INTAKE
PLAN
16′
.771′
9.5° ±
7.729′
30′
6′
ELEVATION
EL 274.33′
EL 269.00′
5.333′
6° ±
3.0′
3.035′R
6′R
8′
6′
EL 277′
LOCATION OF VORTIMETER
OR IMPACT TUBES
PIEZOMETER



LOCATION OF IMPACT TUBES AND PIEZOMETERS TYPE 1 PUMP INTAKE

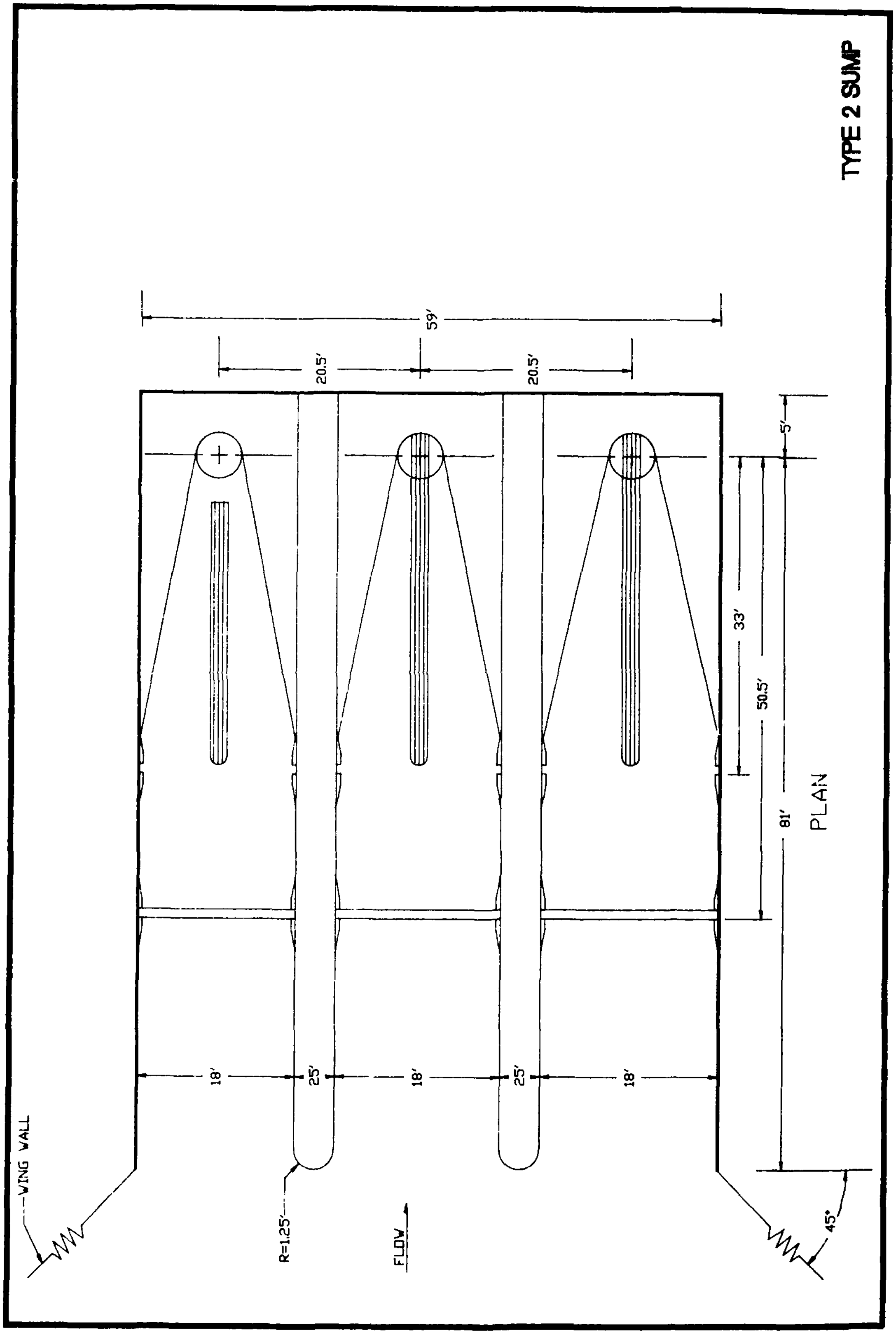


LOCATION OF IMPACT TUBES AND PIEZOMETERS TYPE 2 PUMP INTAKE



TYPE 1 SUMP
59'
20.5'
20.5'
5'
33'
50.5'
81'
136'
18'
25'
18'
25'
18'
PLAN
R = 1.25'
U-SECTION
FLOW
50'
WING WALL
45°

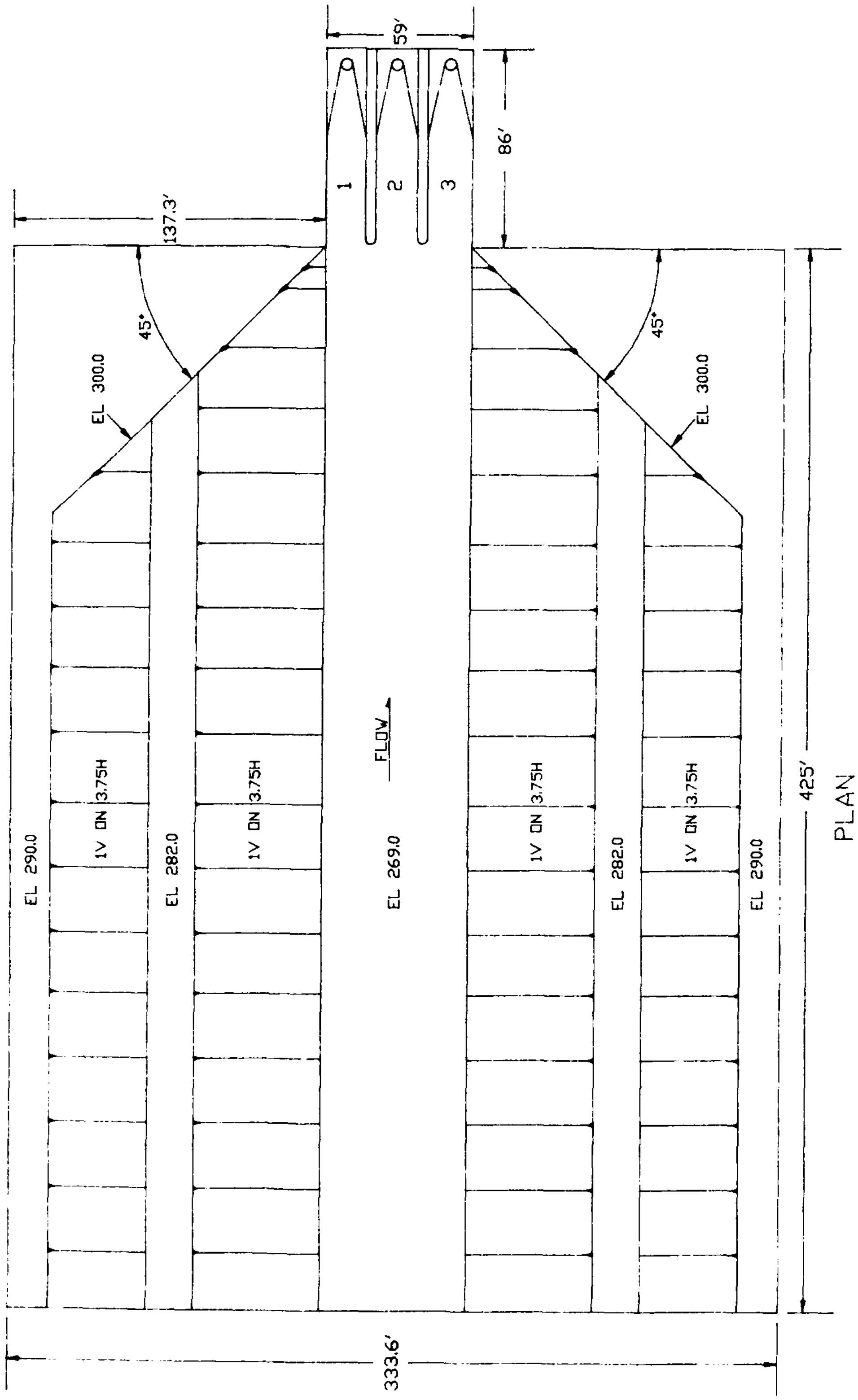
PLATE 4



TYPE 2 SUMP
59'
20.5'
20.5'
5'
33'
50.5'
81'
PLAN
18'
25'
18'
25'
18'
WING WALL
R=1.25'
FLOW
45°

PLATE 5

TYPE 1 INLET CHANNEL
TYPE 2 SUMP



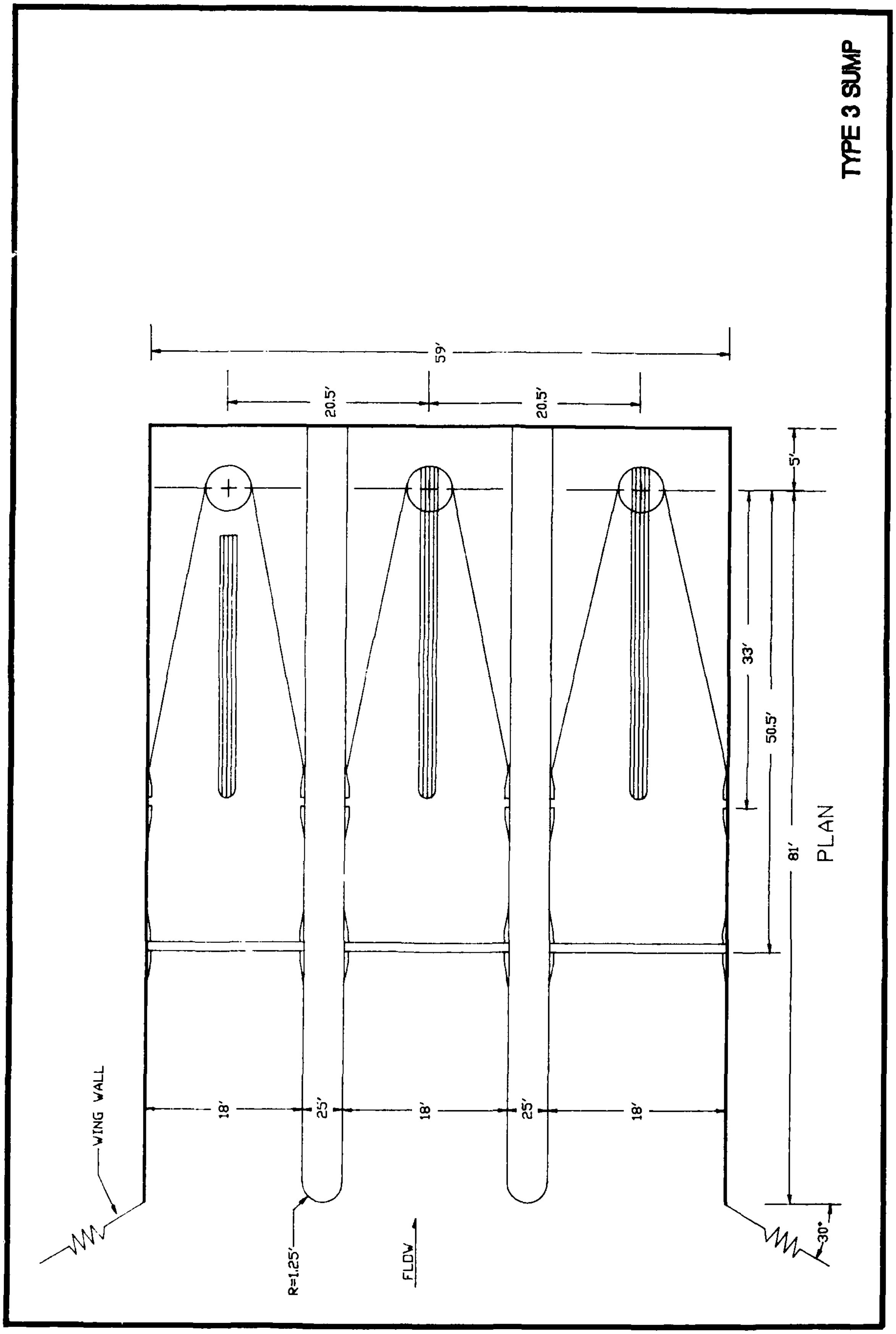
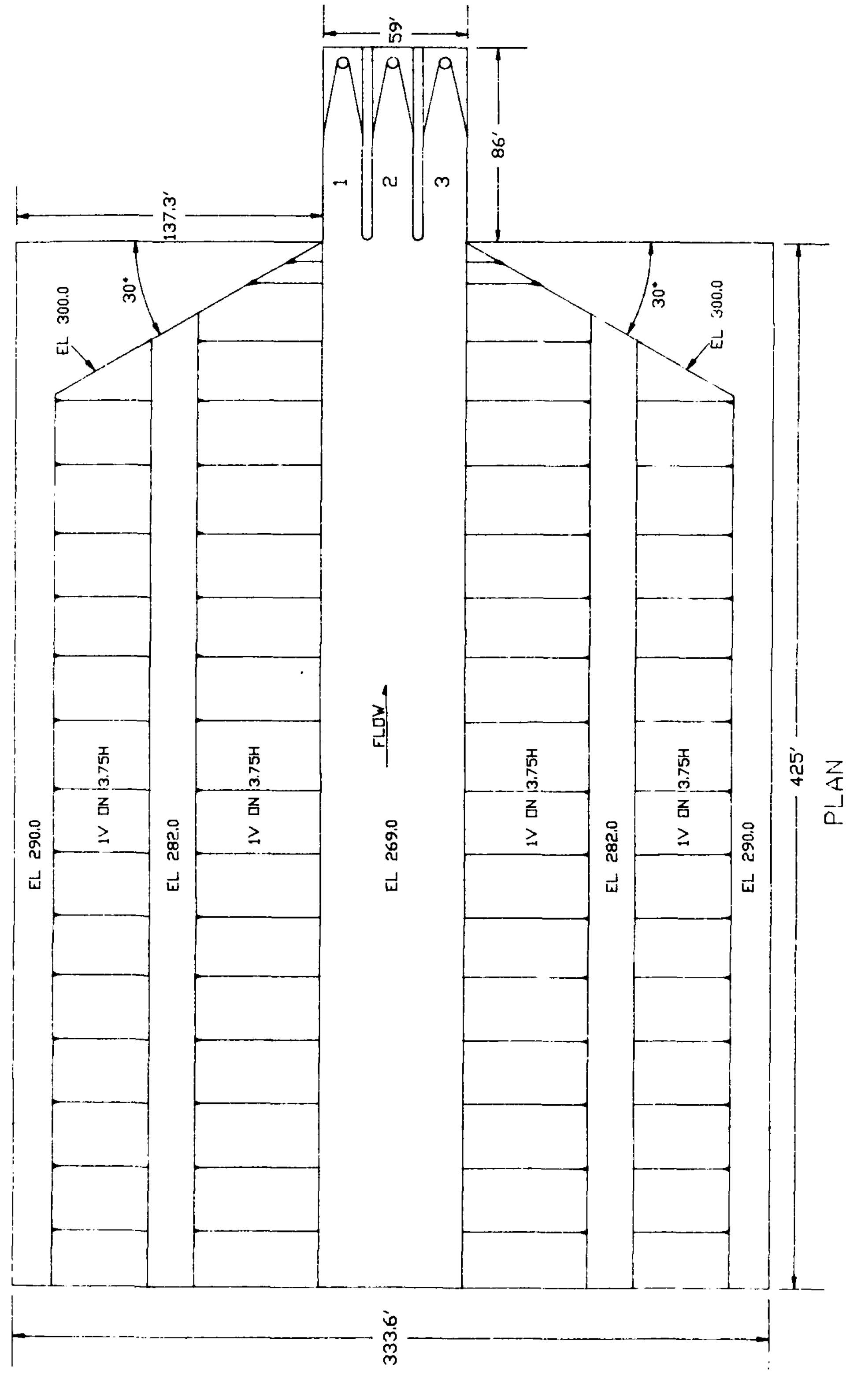
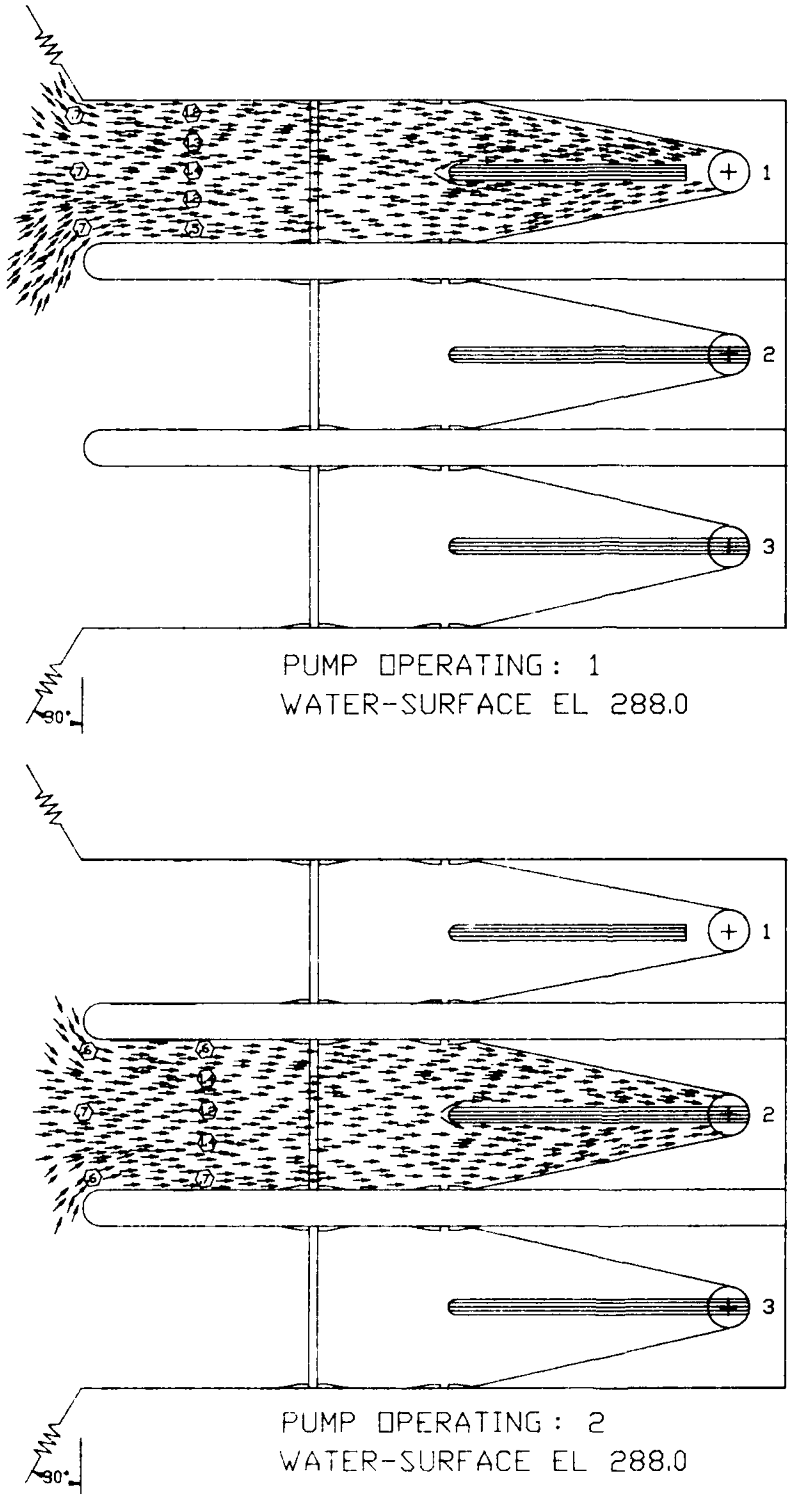

TYPE 3 SUMP
59'
20.5'
20.5'
5'
33'
50.5'
81'
PLAN
WING WALL
18'
25'
18'
25'
18'
R=1.25'
FLOW
30°

PLATE 7



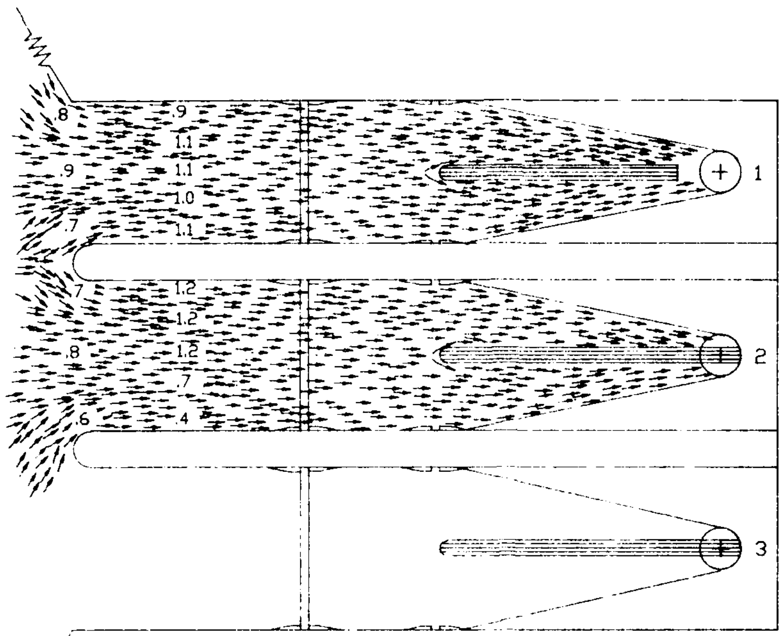
TYPE 1 INLET CHANNEL
TYPE 3 SUMP

PLATE 8

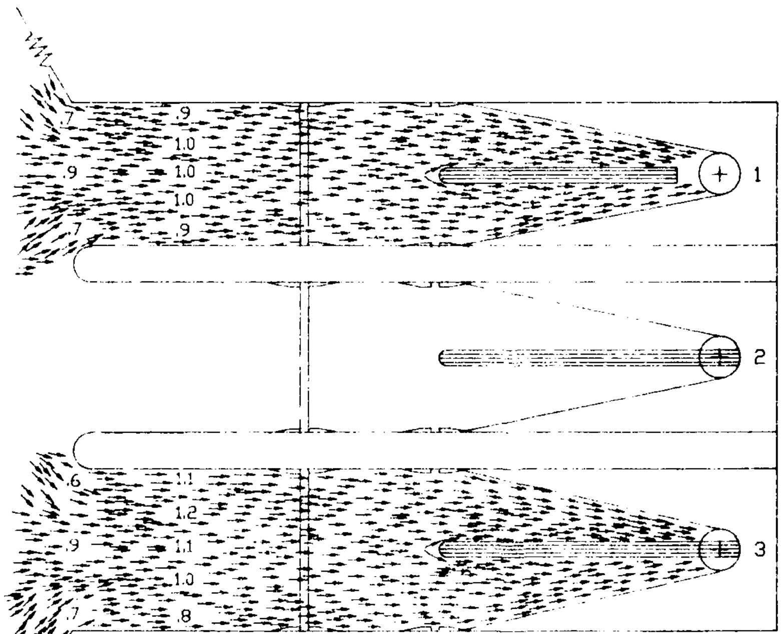


CURRENT PATTERNS AND VELOCITIES
TYPE 1 INLET CHANNEL
TYPE 3 SUMP

NOTE: ⑥ = VELOCITIES IN FEET PER SECOND 1 FT ABOVE BOTTOM



PUMPS OPERATING : 1 AND 2
WATER-SURFACE EL 288.0



PUMPS OPERATING : 1 AND 3
WATER-SURFACE EL 288.0

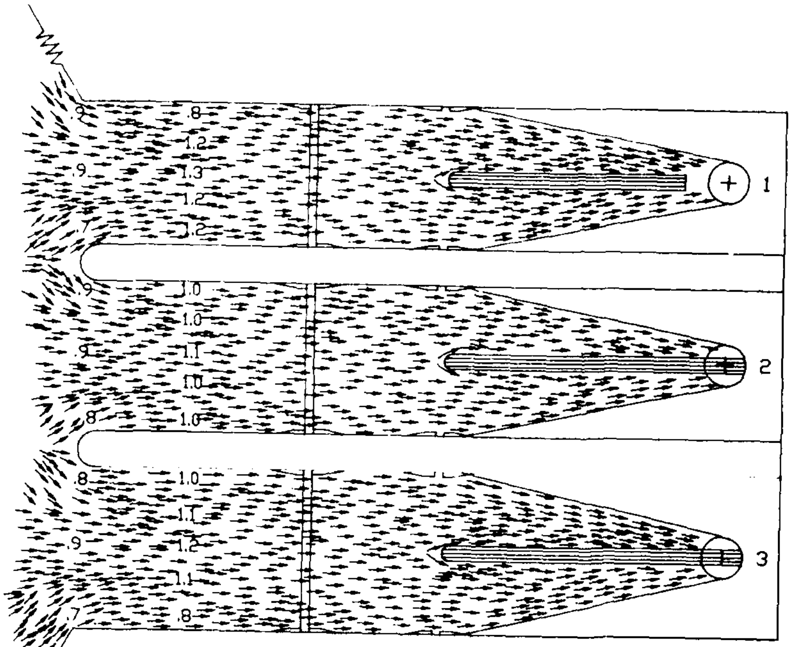
CURRENT PATTERNS AND VELOCITIES

TYPE 1 INLET CHANNEL

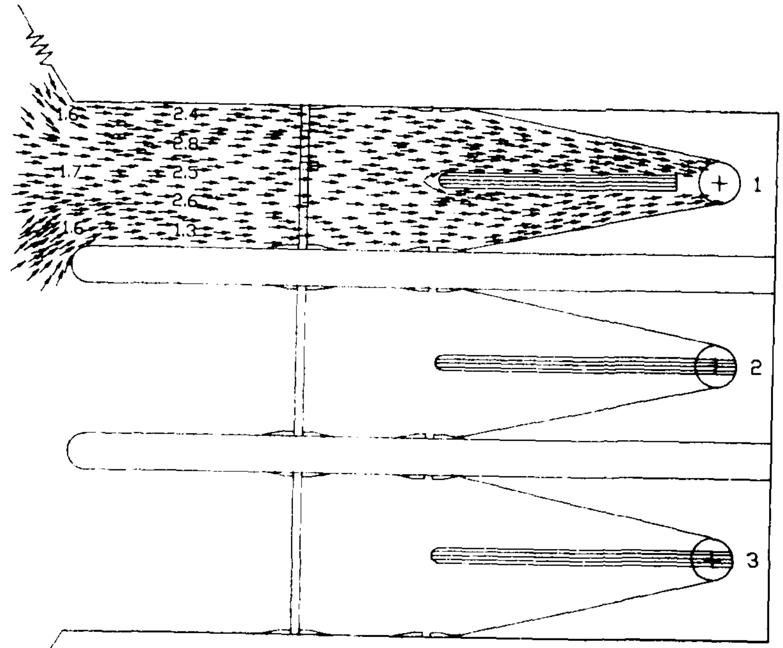
TYPE 3 SUMP

NOTE .6 —VELOCITIES IN FEET PER SECOND ONE FT ABOVE BOTTOM

PLATE 9
(SHEET 2 OF 5)



PUMPS OPERATING : 1,2, AND 3
WATER-SURFACE EL 288



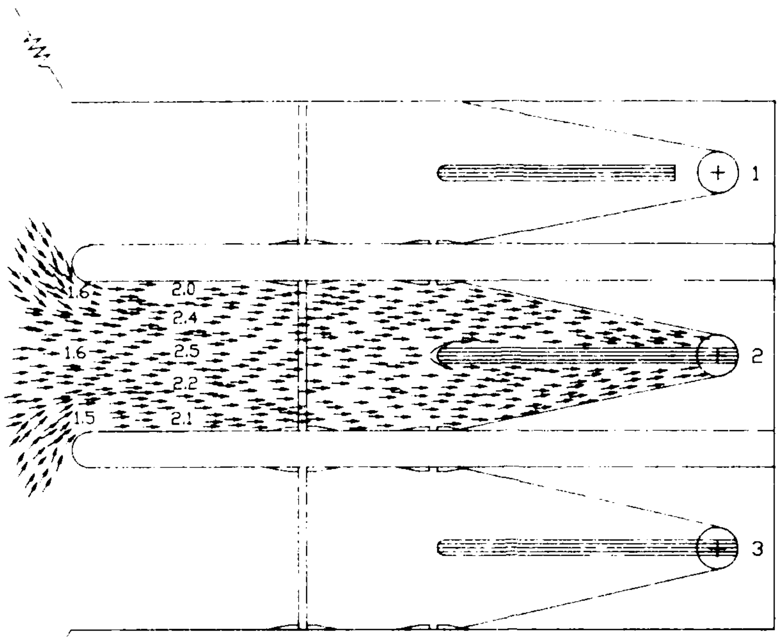
PUMP OPERATING : 1
WATER-SURFACE EL 277

CURRENT PATTERNS AND VELOCITIES

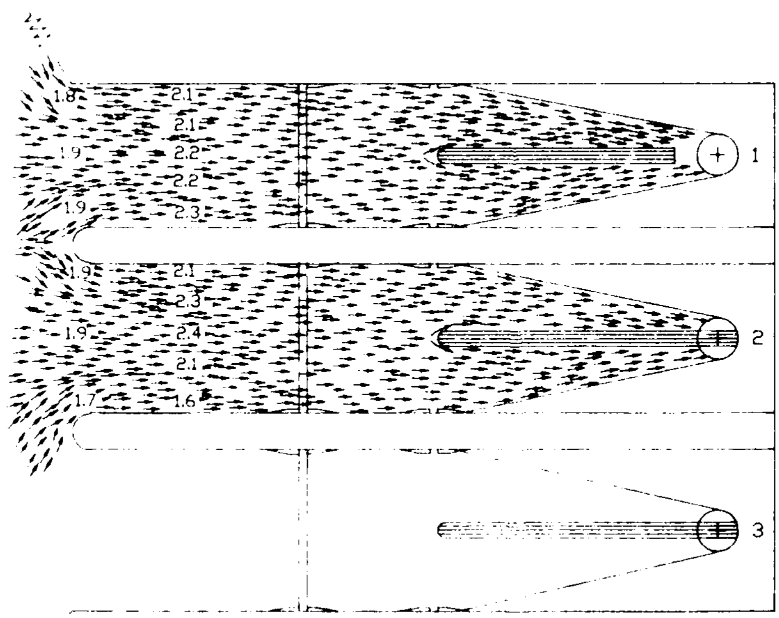
TYPE 1 INLET CHANNEL
TYPE 3 SUMP

NOTE : (.6) = VELOCITIES IN FEET PER SECOND 1 FT ABOVE BOTTOM

PLATE 9
(SHEET 3 OF 5)



PUMP OPERATING : 2
WATER-SURFACE EL 277

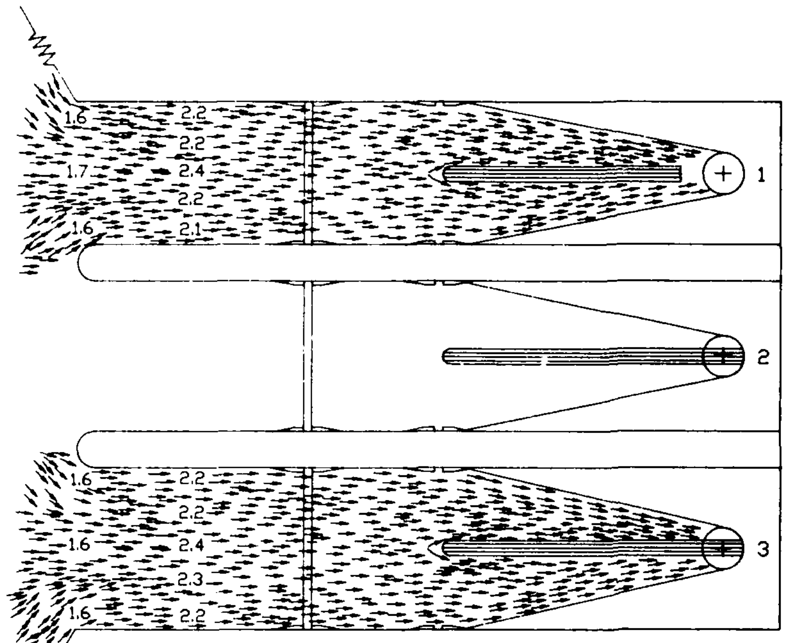


PUMPS OPERATING : 1 AND 2
WATER-SURFACE EL 277

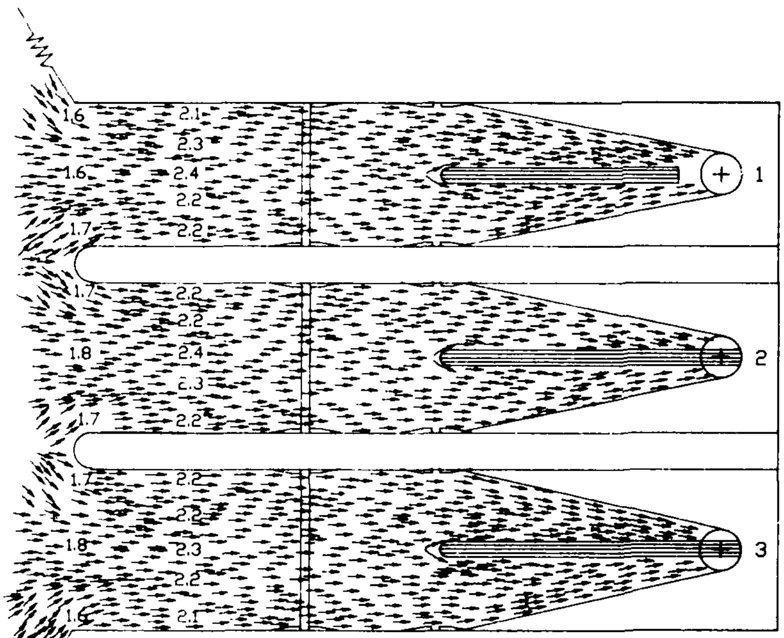
CURRENT PATTERNS AND VELOCITIES

TYPE 1 INLET CHANNEL
TYPE 3 SUMP

NOTE .6 = VELOCITIES IN FEET PER SECOND ONE FT ABOVE BOTTOM



PUMPS OPERATING : 1 AND 3
WATER-SURFACE EL 277



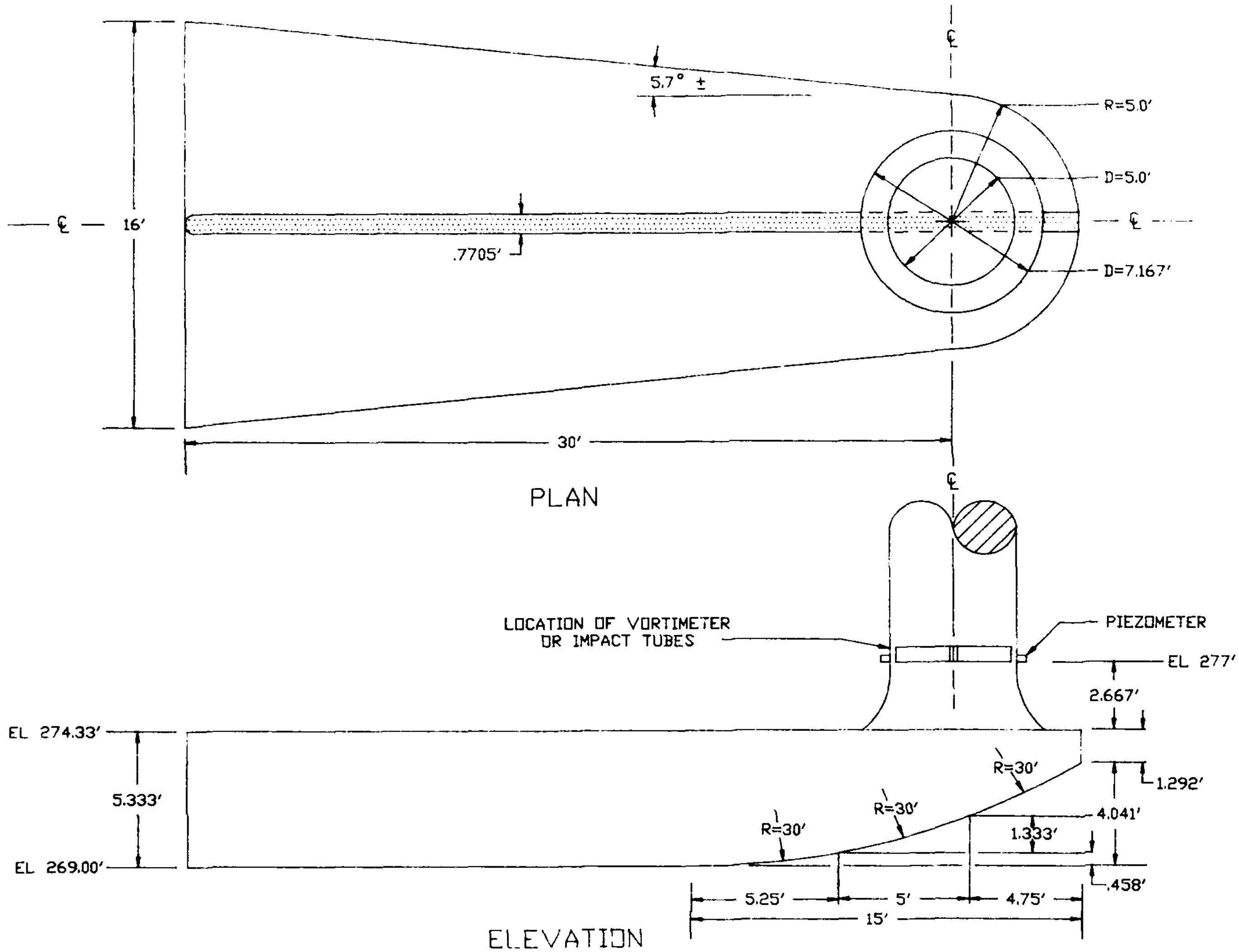
PUMPS OPERATING : 1,2, AND 3
WATER-SURFACE EL 277

CURRENT PATTERNS AND VELOCITIES

TYPE 1 INLET CHANNEL

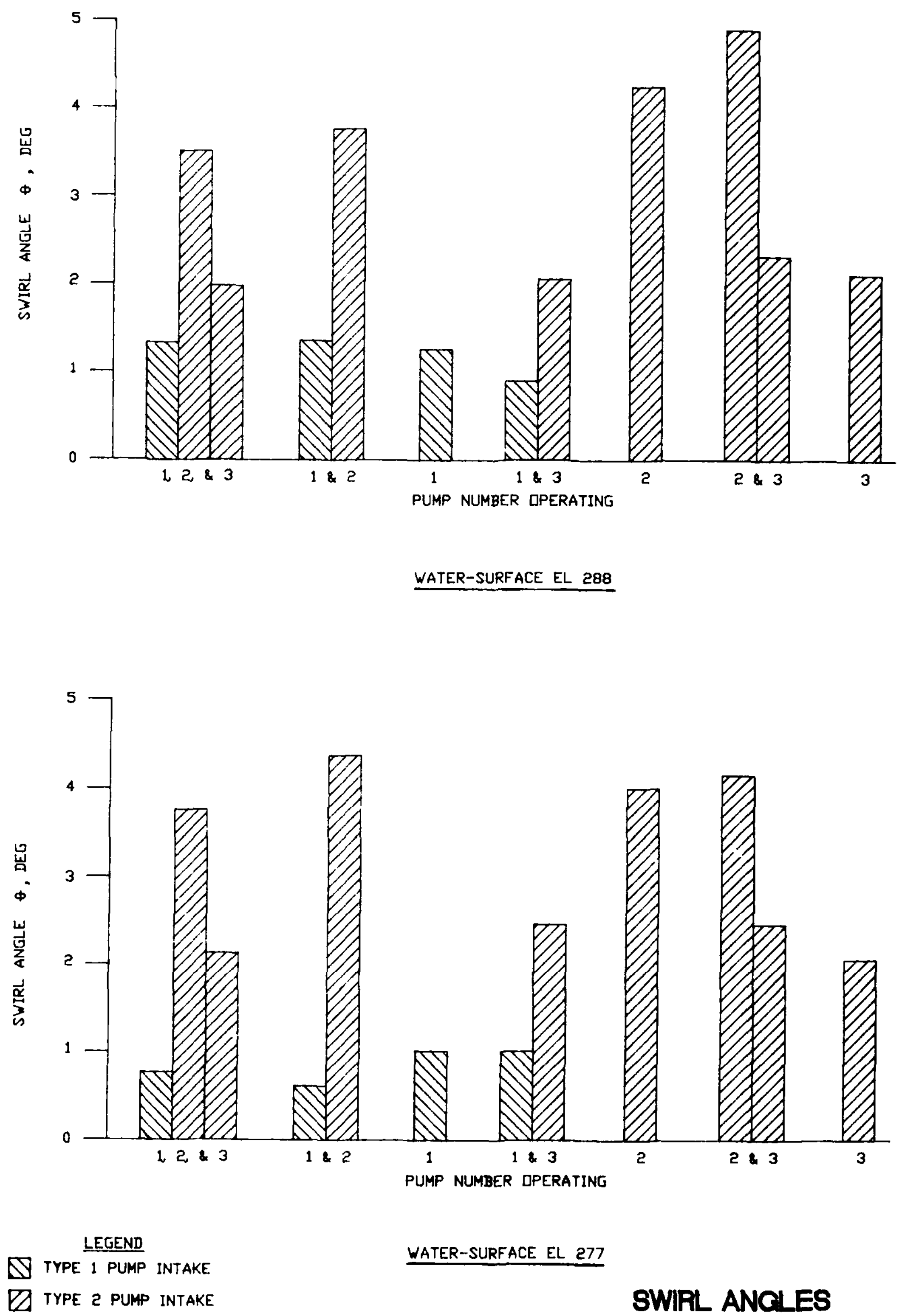
TYPE 3 SUMP

NOTE 6 =VELOCITIES IN FEET PER SECOND ONE FT ABOVE BOTTOM

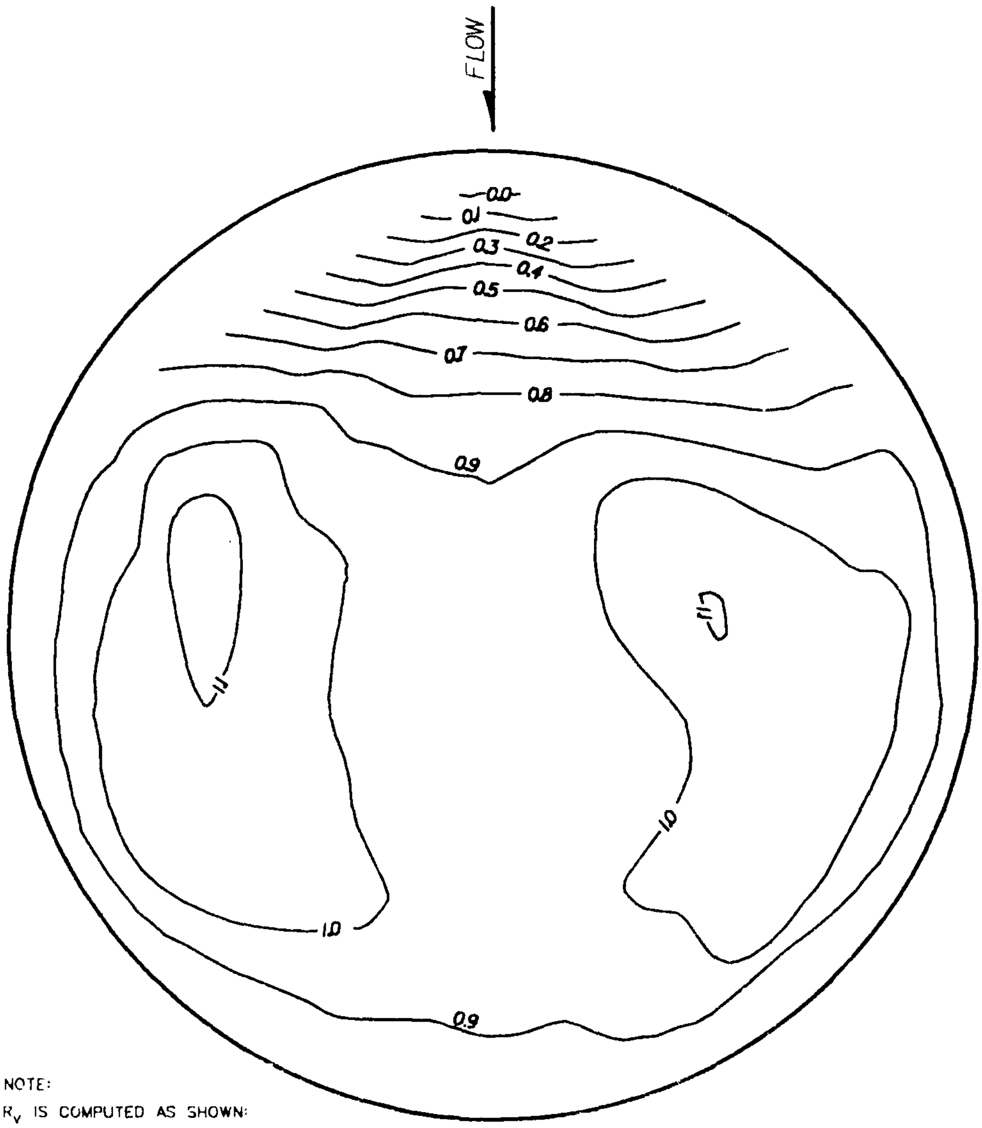


TYPE 2 PUMP INTAKE

PLATE 10



SWIRL ANGLES
TYPE 1 INLET CHANNEL
TYPE 3 SUMP



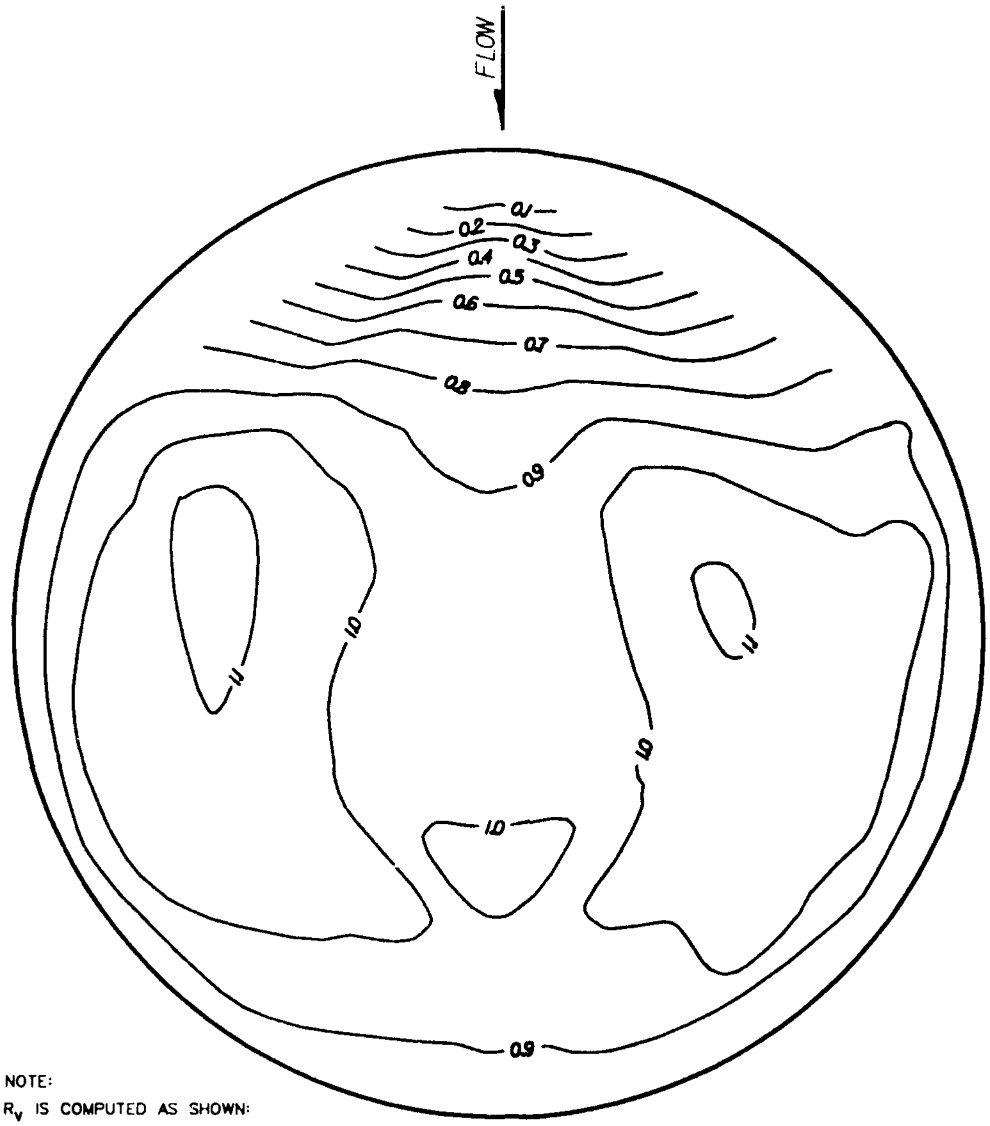
NOTE:

R_V IS COMPUTED AS SHOWN:

$$R_V = \frac{V_M}{V_C} = \frac{\text{MEASURED AVERAGE AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
AVERAGE VALUES R_V
PUMP 1
TYPE 1 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 1



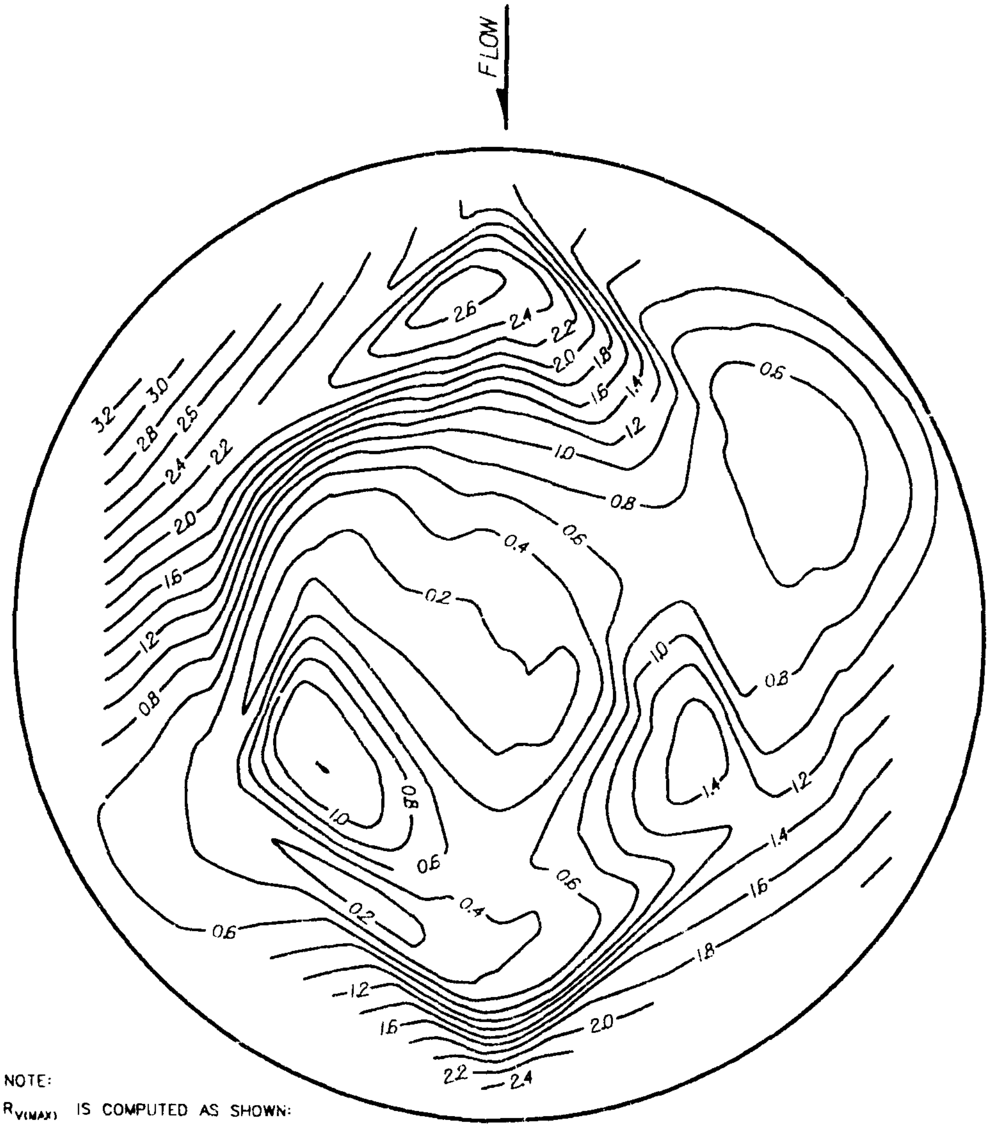
NOTE:

R_V IS COMPUTED AS SHOWN:

$$R_V = \frac{V_M}{V_C} = \frac{\text{MEASURED AVERAGE AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
AVERAGE VALUES R_V
PUMP 1
TYPE 1 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMPS OPERATING: 1, 2, AND 3



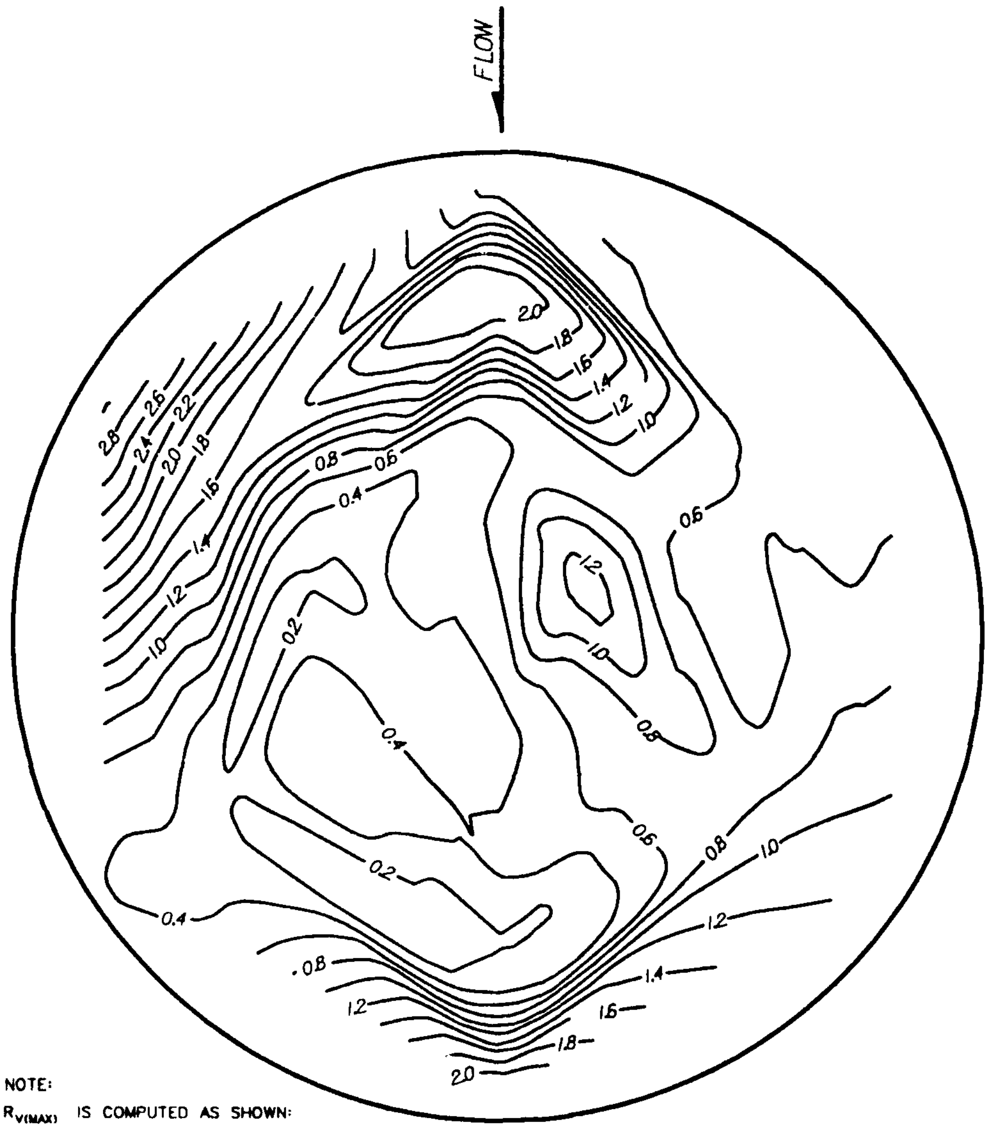
NOTE:

$R_{V(MAX)}$ IS COMPUTED AS SHOWN:

$$R_{V(MAX)} = \frac{V_{M(MAX)}}{V_C} = \frac{\text{MEASURED MAXIMUM AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
MAXIMUM DEVIATION VALUES $R_{V(MAX)}$
BETWEEN 6.62 AND 6.67 SEC
PUMP 1
TYPE 1 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 1



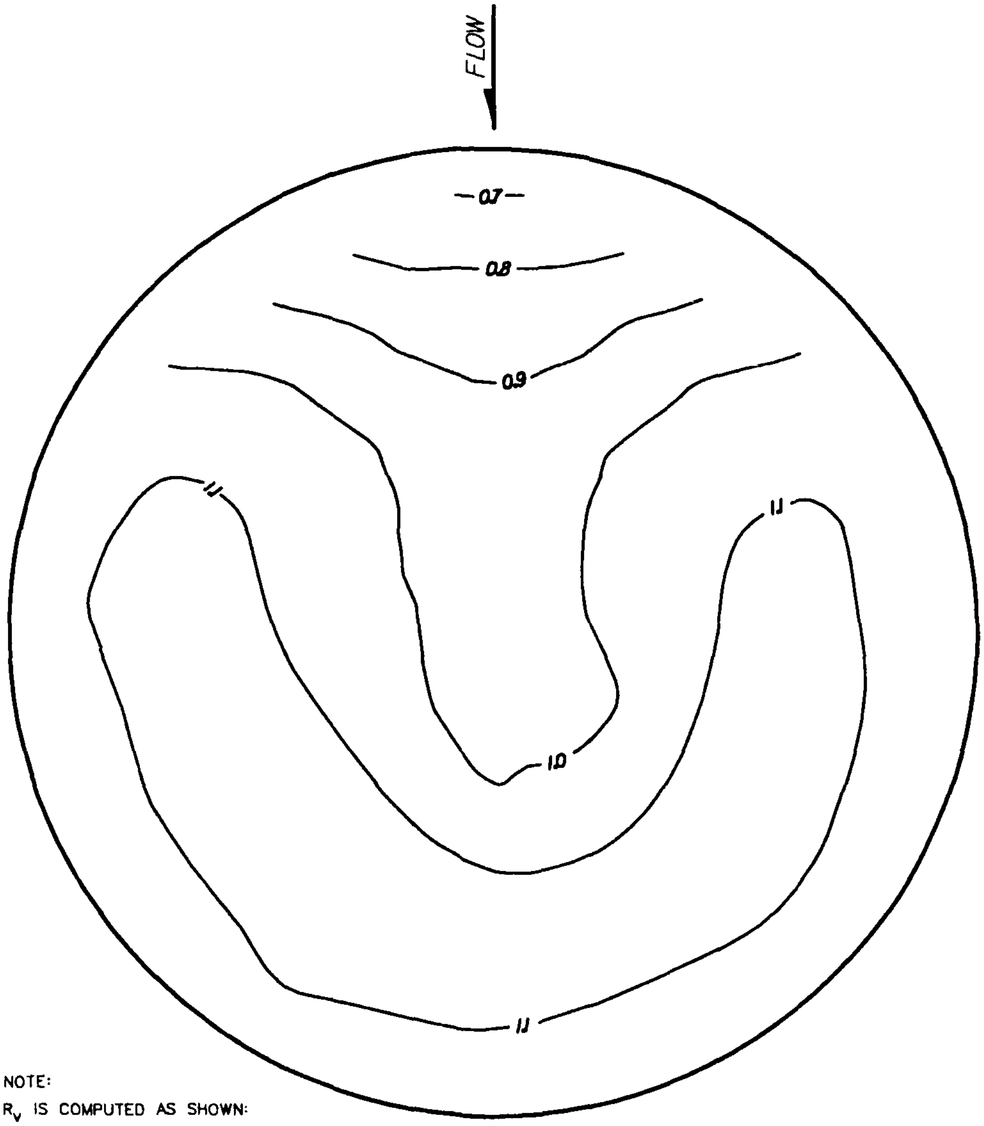
NOTE:

$R_{V(MAX)}$ IS COMPUTED AS SHOWN:

$$R_{V(MAX)} = \frac{V_{M(MAX)}}{V_C} = \frac{\text{MEASURED MAXIMUM AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
MAXIMUM DEVIATION VALUES $R_{V(MAX)}$
BETWEEN 9.96 AND 10.02 SEC
PUMP 1
TYPE 1 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMPS OPERATING: 1, 2, AND 3

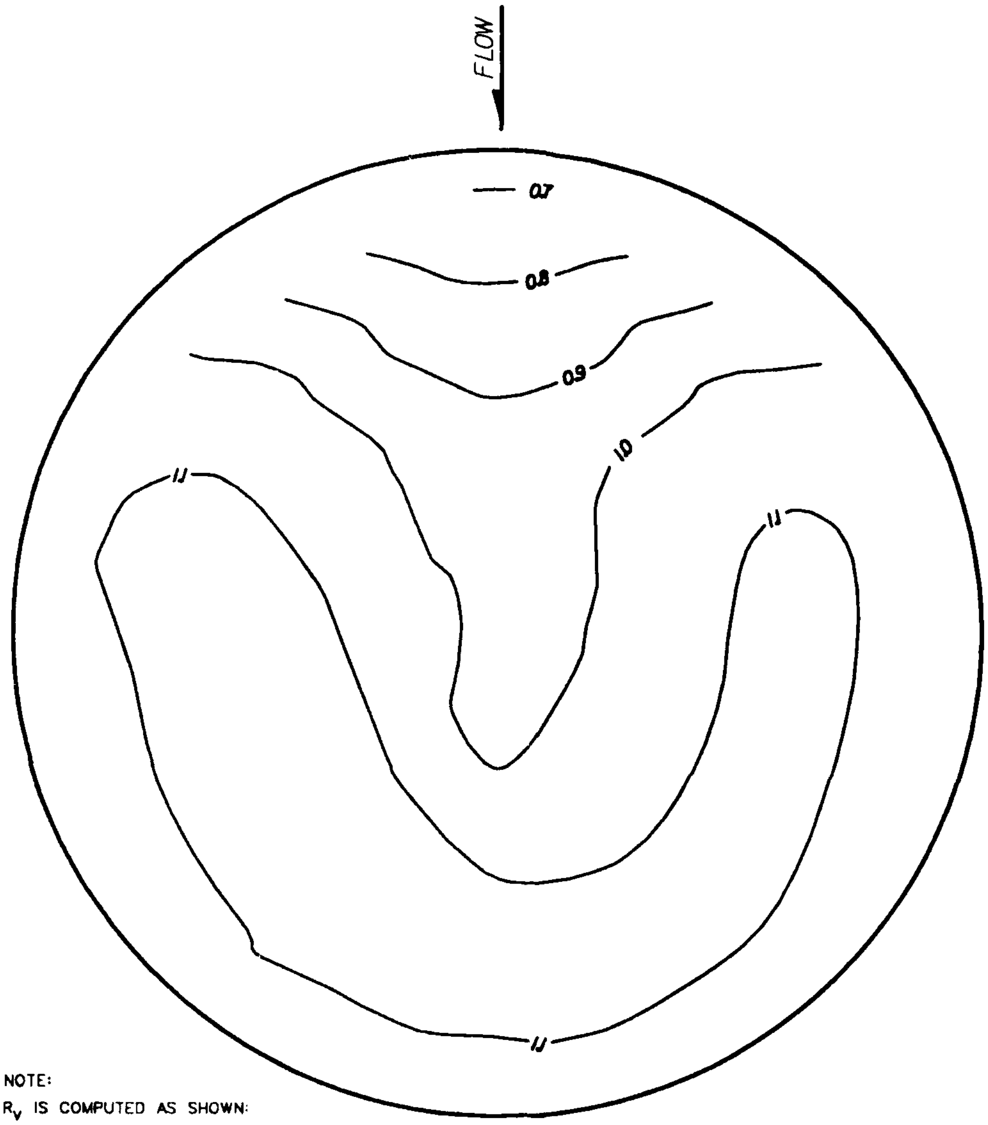


NOTE:
R_V IS COMPUTED AS SHOWN:

$$R_V = \frac{V_M}{V_C} = \frac{\text{MEASURED AVERAGE AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
AVERAGE VALUES R_V
PUMP 3
TYPE 2 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 3



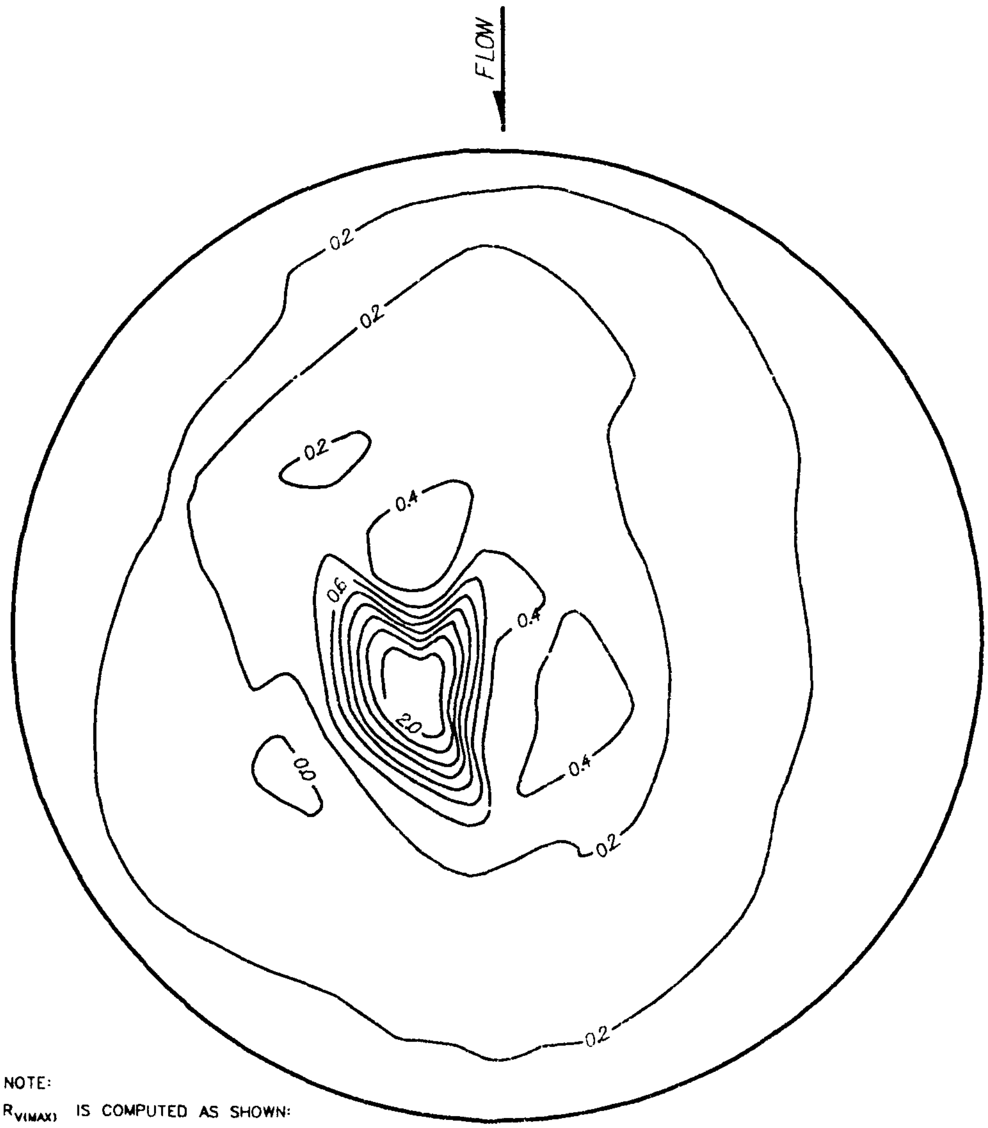
NOTE:

R_V IS COMPUTED AS SHOWN:

$$R_V = \frac{V_M}{V_C} = \frac{\text{MEASURED AVERAGE AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
AVERAGE VALUES R_V
PUMP 3
TYPE 2 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMPS OPERATING: 1, 2, AND 3



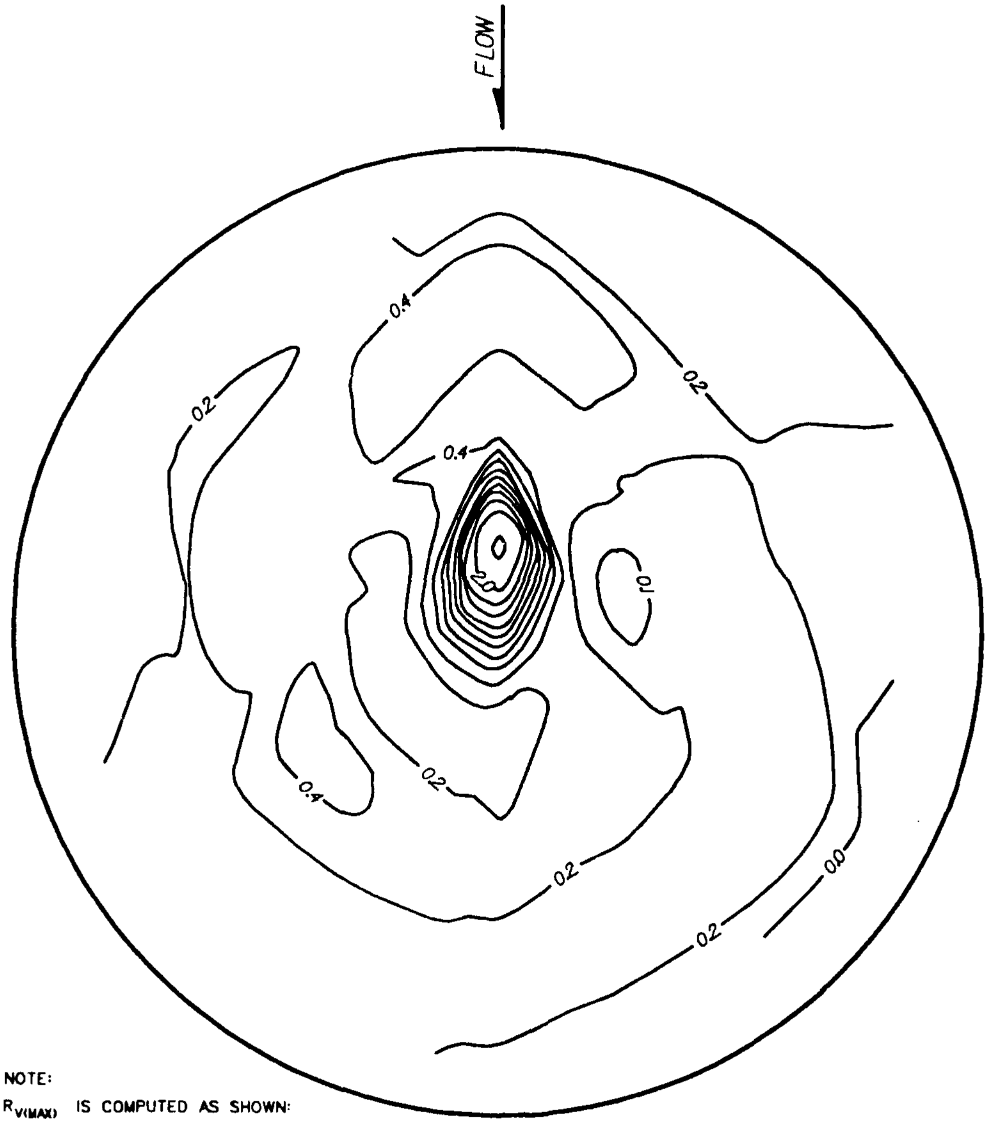
NOTE:

$R_{V(MAX)}$ IS COMPUTED AS SHOWN:

$$R_{V(MAX)} = \frac{V_{M(MAX)}}{V_C} = \frac{\text{MEASURED MAXIMUM AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
MAXIMUM DEVIATION VALUES $R_{V(MAX)}$
BETWEEN 1.62 AND 1.72 SEC
PUMP 3
TYPE 2 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 3



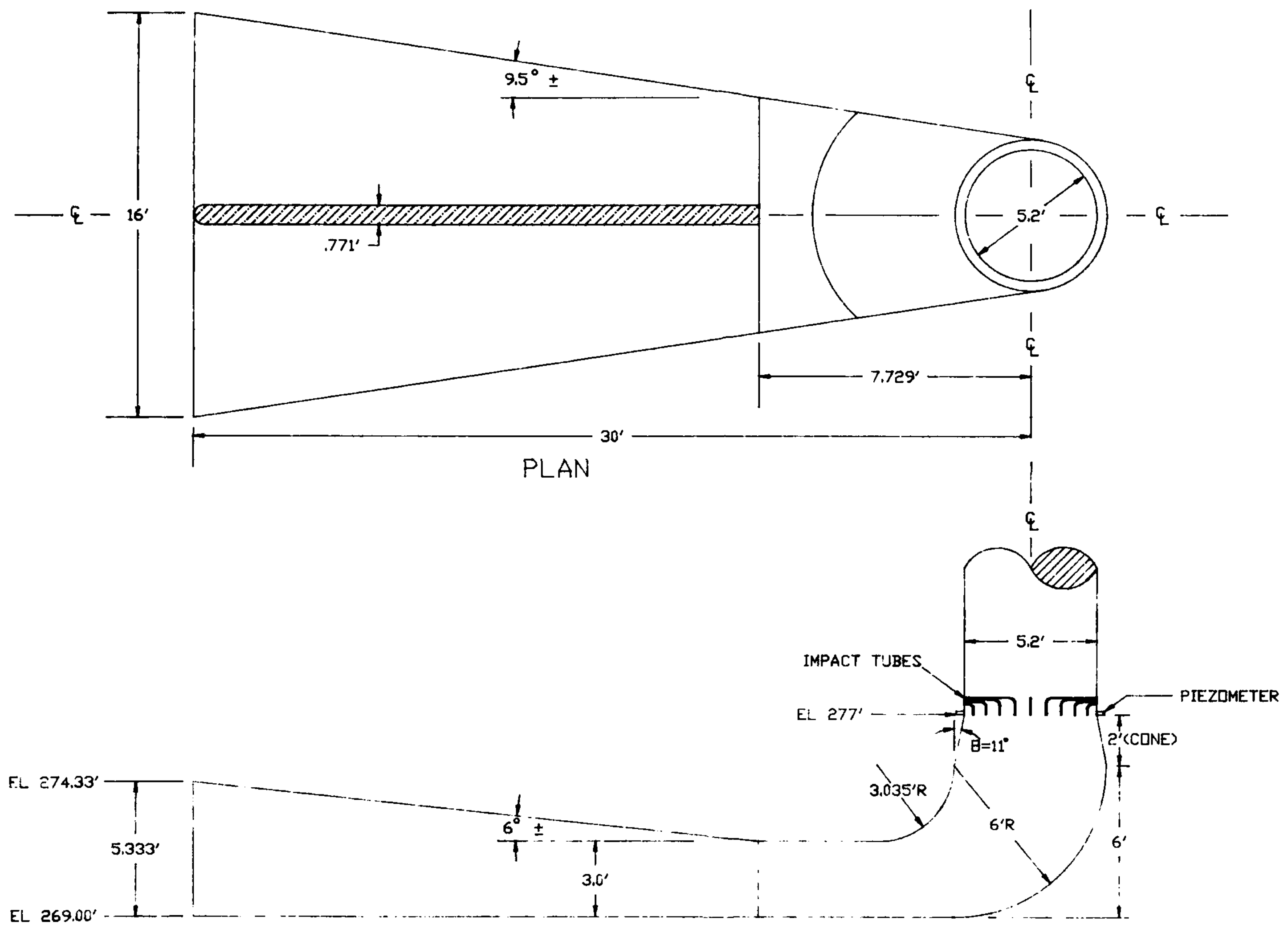
NOTE:

$R_{V(MAX)}$ IS COMPUTED AS SHOWN:

$$R_{V(MAX)} = \frac{V_{M(MAX)}}{V_C} = \frac{\text{MEASURED MAXIMUM AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

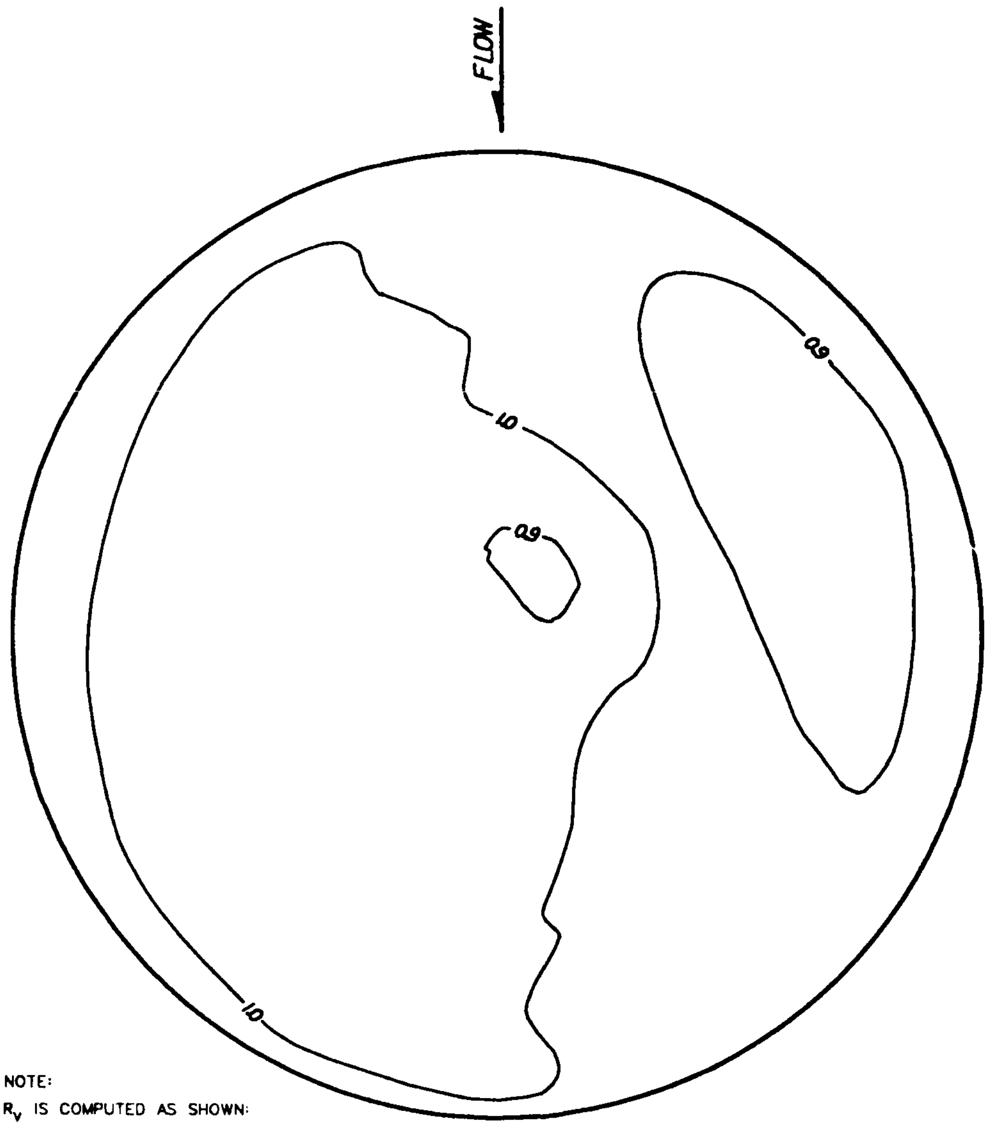
$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
MAXIMUM DEVIATION VALUES $R_{V(MAX)}$
BETWEEN 31.24 AND 31.29 SEC
PUMP 3
TYPE 2 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMPS OPERATING: 1, 2, AND 3



ELEVATION

TYPE 3 PUMP INTAKE



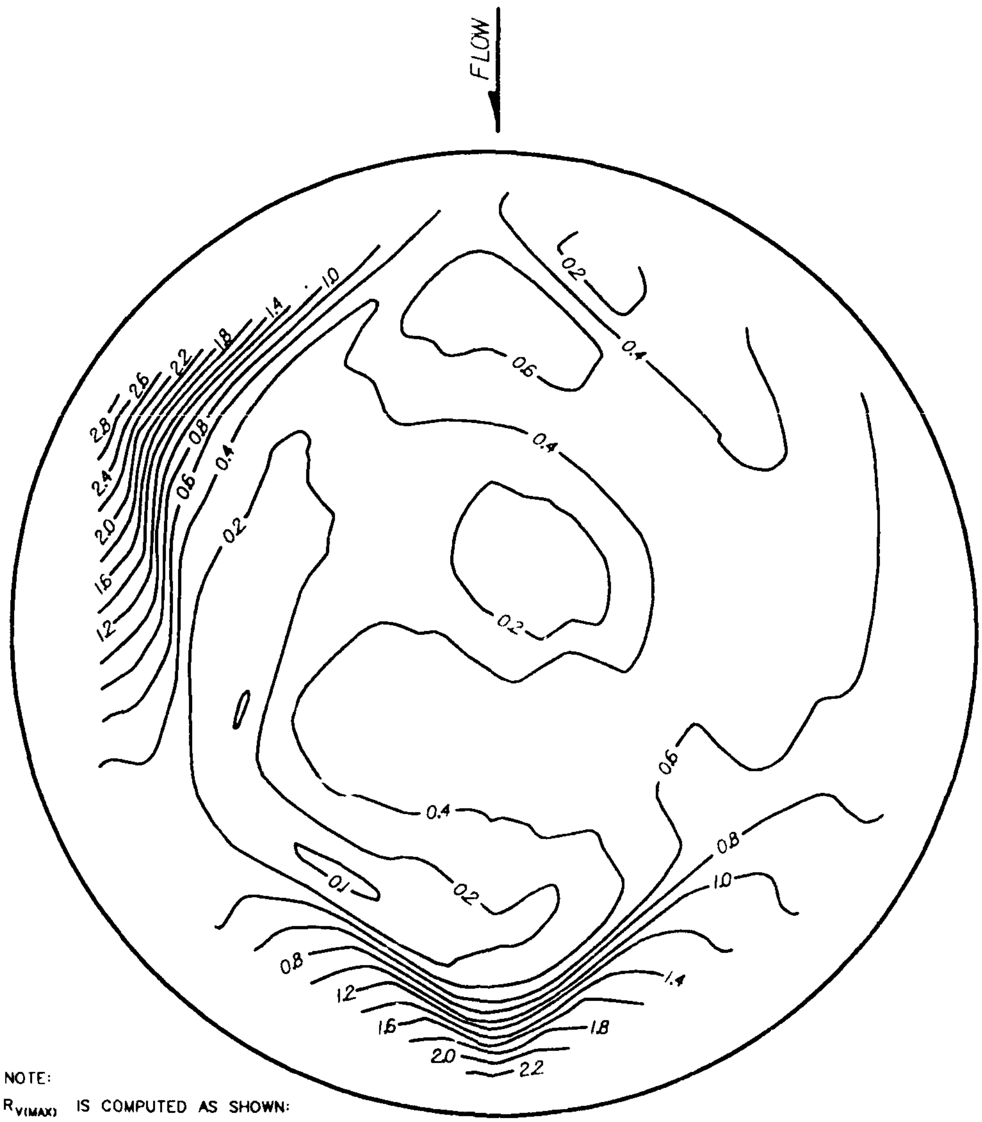
NOTE:

R_V IS COMPUTED AS SHOWN:

$$R_V = \frac{V_M}{V_C} = \frac{\text{MEASURED AVERAGE AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
AVERAGE VALUES R_V
PUMP 1
TYPE 3 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 1



NOTE:

$R_{V(MAX)}$ IS COMPUTED AS SHOWN:

$$R_{V(MAX)} = \frac{V_{M(MAX)}}{V_C} = \frac{\text{MEASURED MAXIMUM AXIAL VELOCITY}}{\text{COMPUTED AVERAGE AXIAL VELOCITY}}$$

$$V_C = \frac{Q}{A} = \frac{\text{DISCHARGE}}{\text{AREA OF PUMP COLUMN CROSS SECTION}}$$

LINES OF EQUAL VELOCITY RATIOS
MAXIMUM DEVIATION VALUES $R_{V(MAX)}$
BETWEEN 21.06 AND 21.10 SEC
PUMP 1
TYPE 3 PUMP INTAKE
WATER-SURFACE EL 277.0
DISCHARGE PER PUMP 333 CFS
PUMP OPERATING: 1