UNIVERSITY OF MINNESOTA
ST. ANTHONY FALLS HYDRAULIC LABORATORY

Project Report No. 208

HYDRAULIC MODEL STUDIES OF THE LAKE AVENUE CONTROL STRUCTURE SITE 45

by

John M. Killen

and

Joseph M. Wetzel

Conducted for

HARZA ENGINEERING COMPANY
Chicago, Illinois

and

DIVISION OF PURE WATERS
Monroe County, New York

March, 1985
Minneapolis, Minnesota

University of Minnesota
St. Anthony Falls Hydraulic Laboratory

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PREFACE

The results of the study of the Site 45 diversion structure model are described in this report and in a 16 mm movie film which shows the flow action on various parts of the model structure.

The model was constructed by the St. Anthony Falls Hydraulic Laboratory shop personnel under the supervision of Jack Vandenberg. The model was operated and data assembled by Christopher Thompson, Timothy O'Mara, and Rienk DeVries. Movies were made under the supervision of Christopher Thompson and Timothy O'Mara, and photography was by Karl Wikstrom. Final editing and preparation of the report was done by Patricia Swanson.

TABLE OF CONTENTS

LIST OF PHOTOGRAPHS

Photo No.

LIST OF FIGURES

Chart No.

Hydraulic Model Studies of the

Lake Avenue Control Structure, Site 45

I. INTRODUCTION

The tunnel system for the Combined Sewer Overflow Abatement Project (CSOAP) for the City of Rochester, New York, requires numerous control structures. Site 45 is the designated name and location of one such control structure. It is located on the west bank of the Genesee River and receives flow from the Lake Avenue Tunnel and the Tiger Carlisle Tunnel.

The Site 45 control structure performs three functions. First, it contains a centrally located chamber which provides relief from waterhammer and surge pressures that will occur in the tunnels as a result of stormwater inflows. Second, if the inflow volume is great enough, the structure provides overflow relief by directing excess stormwater to two dropshafts which lead to the Genesee River below. Third, the structure controls the rate of flow through two parallel conduits which cross over to the east side of the Genesee River, and to additional structures located downstream, including the Frank E. Van Lare Sewage Treatment Plant.

The control structure at Site 45 is designed to pass up to 375 cfs across the Genesee River to the Van Lare Treatment Plant. Flows greater than 375 cfs will exceed the capacity of the treatment plant and will be directed via the overflow relief to the Genesee River. The design maximum inflow to the Site 45 structure is 3000 cfs.

Control of the rate of flow to the sewage treatment plant will be accomplished by means of control gates within the Site 45 structure. The gate openings will be set automatically to pass a given flow with a range of head differences. These head differences are dependent on the water surface elevation in the surge chamber within the structure and the head required to establish a specific flow to the sewage treatment plant.

A model of the Site 45 structure was built at the St. Anthony Falls Hydraulic Laboratory from drawings of the proposed structure supplied by Harza Engineering Company (Dwg. 1330 HYD 4500 R1). Figures 1 and 2 show the plan and elevation of the structure obtained from these drawings. Photographs of the model are shown in Photos 1 and 2. The details of the specific parts of the model will be explained as the functions of the various components are discussed.

Froude law scaling was used to establish dynamic similarity between the model and prototype, as gravity is the dominant force producing motion. The following expressions were used to convert the geometric, kinematic, and dynamic quantities from the model to the prototype. The subscripts m, p, and r refer to model, prototype, and ratio of model scale to prototype scale, respectively.

Quantity	Ratio r	Scale
Length L	$L_r = L_m/L_p$	1:16
Discharge Q	$Q_r = L_r^{5/2}$	1:1024
Velocity V	$V_r = L_r^{1/2}$	1:4
Pressure P	$P_r = L_r$	1:16
Time T	$T_r = L_r^{1/2}$	1:4

The scale ratio of 1:16 was chosen to provide adequate dynamic similarity to the prototype, and to utilize available construction materials and laboratory space.

II. CONCLUSIONS

The following comments are made with reference to the five objectives set forth in Section IV.

1. Open-channel flows of less than 375 cfs were found to be confined to the molded channel in the floor of the structure. This verifies the channel design selected. The model drained very well upon shutdown, leaving no pools of standing water.

2. Head-discharge curves were obtained for various settings of the head-breaker gates under steady flow conditions. Some weak vortices were occasionally observed in the surge chamber above the gate location. These vortices did not draw air, and occurred in a random fashion for only a very short period of time. It is expected that vorticity should not be a serious problem in the prototype structure.

3. The flow velocity from the control gates can be kept below 28 fps for a discharge of 375 cfs with the proper gate opening and choice of gate location.

4. Water surface elevations in the surge chamber were measured for various inflow conditions with the head-breaker gates closed. The inflow from each of the two 14 ft diameter inlet conduits was varied to provide a difference of discharges ranging from 0 to 1500 cfs. The maximum lateral variation in water surface elevation was about 3.5 percent.

5. The flow in the circumferential collection channel and overflow conduits was found to be subcritical for the maximum flow of 3000 cfs, with the control section located near the entrance to the dropshaft.

III. DESCRIPTION OF THE MODEL

The operation of the model can be most easily visualized by reference to the plan and elevation of the structure shown in Figures 1 and 2. The flow enters the structure through two 14 ft conduits with an invert elevation of 348.4 ft. These conduits are shown in Photo 3. The design maximum flow in each conduit is 1500 cfs. This flow is monitored in the model by calibrated orifice meters in each of the 14 ft lines and controlled by gate valves as shown on Photo 1.

A. Surge Chamber

In plan, the surge chamber is composed of a semicircular section, with a 29 ft radius, and a rectangular 29 x 58 ft section. The sidewalls of the model surge chamber were constructed of transparent plastic to permit viewing and lighting of the flow. The top of the surge chamber wall has the form of an overflow weir with a crest elevation of 388 ft. The weir crest and surge chamber is shown in Photos 4 and 5. The carriage shown in Photo 5 above the surge chamber was used to support a point gage for measurement of water surface elevation. The assembly was movable in both longitudinal and lateral directions. The metal strip along the top of the weir, which is visible in the photographs, is for correction of any irregularities in the weir elevation in the model and is not expected to exist in the prototype. A contoured floor as shown in Figures 1 and 2 was constructed of concrete and was installed in the surge chamber, head-breaker-transition, chamber and circumferential collection channel. The possibility of septic water pooling at some point in the structure during low flows is reduced to a minimum by the shape of the contoured invert.

B. Circumferential Overflow Collection Channel

The overflow from the surge chamber passes into a circumferential side channel around the outside periphery of the surge chamber. A vertical wall placed along the longitudinal axis of the structure divides the overflow into two parts. The channel is rectangular in cross section with a 14 ft width at the upstream end and an 18 ft width at the junction with the 18 ft diameter conduits leading to the dropshaft structures. The flat bottom is set on a one percent slope. Fillets are installed at the lower corners of the channel.

For the design discharges, the flow depth in the 18 ft diameter conduits was not expected to exceed 9 ft. Therefore, for ease of fabrication and measurement of water surface profiles, these conduits were constructed of sheet metal with a semicircular cross section. Photo 2 shows the semicircular troughs used to simulate the 18 ft circular conduits.

C. Head-Breaker Chamber

Downstream of the surge chamber is a head-breaker chamber which is to dissipate the energy of the jets spouting from the gates installed in the vertical wall separating the surge chamber and the head-breaker chamber. Three sets of gates (Photo 4) were installed to control the flow from the surge chamber for levels in this chamber from the lower gate centerline at El. 349.41 ft to above the weir crest at El. 388 ft.

D. Transition Chamber

The head-breaker chamber is followed by a transition chamber (Photo 6) which is used to remove any remaining air from the water and to pass the discharge into two 6 ft conduits which lead across the Genesee River. The total flow through the head-breaker chamber, transition chamber, and 6 ft conduits is limited to a maximum of 375 cfs.

E. Model Measurement and Control

Two butterfly valves in the 6 ft conduits were used to control the flow from the transition chamber. A number of head discharge curves were simulated in the model.

Calibrated orifice meters (Photo 1) installed in the 14 ft supply lines were used to measure the inflow to the model. Four boxes fitted with V-notched weirs were used to measure the outflow from the model; two were placed at the discharge end of the 18 ft conduits and two at the discharge of the 6 ft conduits.

IV. OBJECTIVES OF THE MODEL STUDY

The following objectives for the model study have been established by Harza Engineering Company:

(1) Establish acceptable flow conditions throughout the structure during normal low-flow of from 0-375 cfs.

(2) Establish head-breaker gate settings required to pass flows of 150 cfs and 375 cfs during steady-state operation with surge chamber water surface elevations at 5 ft intervals between structure invert elevation and the overflow relief weir elevation of 388 ft.

(3) Establish acceptable flow conditions in the overflow collection channel and the transition chamber for the discharge and conditions of (2) above.

(4) Establish acceptable surge-chamber flow conditions such that the distribution of flow over the overflow relief weirs is steady and uniform. Overflows are to be in 500 cfs increments from 0 to 1500 cfs.

(5) Establish collector channel geometry such that the spatially varied flow in the channel is steady and subcritical with channel HGL below El. 394 ft and with control at dropshaft entrance brink for each flow in (4) above.

V. RESULTS

A. Surge Chamber

The surge chamber must accommodate a great range of flows. Flow less than 375 cfs passes directly through the chamber in channels moulded into the bottom. Flow through the model is shown in Photo 7 for a discharge of 187.5 cfs from each conduit and again at 75 cfs in each channel in Photo 8. A crossover flow from one channel to the other occurs for inflows above 100 cfs in each channel. Photo 9 was taken immediately after shutoff of the inflow and shows drain-down of the internal channels. The model will drain completely when the lower gate on the surge chamber and head-breaker chamber gates are open.

Photos 10 to 21 taken after data collection show flow over the surge chamber weir crest in various combinations of 500 cfs increments of inflow from the 14 ft conduits for a range of 0 to 2000 cfs. The discharges shown in the photos are nominal values. Extreme unbalanced conditions of zero flow from one 14 ft conduit, and 1500 cfs inflow from the other, showed a maximum outflow difference of 200 cfs between each half of the weir.

Measurements of water surface profiles in the surge chamber are shown in Figures 3 through 12 at various distances from and parallel to the head breaker wall for various combinations of flow from 0 to 2000 cfs inflow from the 14 ft conduits. A maximum difference in elevation of one foot can be seen in the profile measured at 8 ft from the head-breaker wall for an extreme unbalanced inflow of zero from one 14 ft conduit and 2000 cfs from the other. This difference in elevation from right to left can be seen to reduce to less than 1/2 ft as the measured profiles are taken further from the downstream wall.

Flow splitters, or "aeration blocks," 4 ft square and 5 ft high, were later installed on the weir crest as shown in Photo 22 to provide adequate ventilation of the weir flow. Figure 12 shows the water surface elevation with the weir aeration blocks installed. Aeration of the weir nappe in the model before aeration blocks were installed was from the end of the weir at the point where the flow passed into the 18 ft conduit at the head-breaker wall.

B. Head-Breaker Chamber

The head-breaker chamber is intended to dissipate energy in the flow from any of six control gates. The maximum opening of the control gates is 4 ft wide and 5.5 ft high and are installed in the wall between the surge chamber and the head-breaker chamber. Two larger isolation gates, each with maximum opening of 7 ft wide and 8 ft high, release the water from the head breaker chamber into the transition chamber. The location of the control gates can be seen in Photo 4, and the downstream isolation gates are shown in Figure 1 and Photo 6. A limit in velocity of 28 fps is imposed on the flow from the control gates by opening only the gates at the proper elevation. The lower two sets of gates are spaced 12.75 and 13.09 ft apart in elevation, respectively, so that this limit can be achieved. Photos 23 through 30 show the jet discharge from the gates located at various levels for different gate openings and surge chamber water surface elevations.

At the time of the tests, the head in the headbreaker chamber was not known as it was to be determined from the resistance and backwater conditions in the 6 ft diameter exit conduits. Therefore, it was intended to establish the gate rating curves with 5 ft differential head increments across the gate. During the initial stages of the testing, it was found most convenient to adjust the flow through the 6 ft conduits with the butterfly control valves to a specified value, and allow the water depth in the head breaker chamber to adjust itself. The inflow was set at a value greater than the flow through the gate so that some water was being discharged over the weir. For most of these test runs, the jet from the gate was submerged. There appeared to be little difference between the discharge coefficients for the submerged and free jets.

Data were taken for various openings of the top, middle, and bottom gates. The measurements were made for the flow through a single gate up to the maximum of 375 cfs. Gates were sized to provide a steady design flow to the treatment plant for as much time as possible during dewatering under falling head conditions. It was found that the selected gate size would easily pass 375 cfs, even at pasrtial openings.

The experimental data for the lower, middle, and top gates are plotted in Figures 13, 14, and 15 for various gate openings. The data were taken with two inlet discharges into the surge chamber. Open symbols are data points for a total balanced inflow of 500 cfs, and the filled symbols are data points for a total balanced inflow of 3000 cfs. Balanced inflow refers to equal discharges from each of the two 14 ft inlet conduits. It should be noted that for the middle and top gate, the maximum opening height is 5.33 ft rather than 5.5 ft. This was found to be a construction error, and since it would not significantly influence the experimental results, the opening was not enlarged for the model tests.

The data were reduced in terms of a discharge coefficient, defined as

$$C_D = \frac{Q}{A\sqrt{2gH}}$$

where Q = discharge through single gate, cfs

A = gate opening area, width x height, ft^2

H = head difference across gate, ft

The discharge coefficients are plotted in Figure 16 as a function of the Reynolds number, which was defined using the hydraulic radius of the gate opening as the characteristic length, or

$$Re = \frac{VR}{\nu} = \frac{Q}{2(w+x)\nu}$$

where w = gate width, ft

x = gate opening, ft

ν = kinematic viscosity, ft^2/sec

V = flow velocity, fps

R = hydraulic radius, ft

The model Reynolds number for the Froude scaled discharge is less than the prototype Reynolds number by a factor of 64. At low values of the Reynolds number, the discharge coefficient was sensitive to the inflow rate, with a lower value generally being noted at the larger inflow rates. As the Reynolds number was increased, the discharge coefficient also increased and approached a constant value. This asymptotic value was slightly different for the case with the gates full open than for partially open gates, with average values of 0.59 and 0.65, respectively.

For the gate in the full open position, the upper nappe of the jet springs freely from the upstream face of the opening in the head-breaker wall rather than from the lip of the sluice gate, which is mounted on the downstream face of the head-breaker wall. This causes a greater contraction in the jet issuing from the gate and thus gives a lower value for the discharge coefficient. For partial gate openings, the upper nappe springs from the sluice gate lip and results in a smaller contraction of the jet, and subsequently larger values for the discharge coefficient. The geometry of the lower gate is nearly that of a sluice gate in a rectangular channel. As the inflow to the surge chamber was increased, the discharge coefficient for full gate opening also increased slightly. Henderson (1966)[1] shows a limiting value of the coefficient of 0.6. The jet issuing from the two inlets apparently gave an effective head differential in excess of the values obtained from the two pool elevations. For the present purpose it was decided to use the previously averaged values for C_D of 0.59 and 0.65 as values which may be typical full scale values for sharp-edged gate openings.

The solid lines plotted in Figures 13, 14, and 15 are the head-discharge curves using the above average values for C_D to show the fit with the experimental data. Curves are plotted in Figure 17 for all gates at even increments of gate opening.

An intermittent separation zone was observed visually on the surface of the lower lip on the middle gate. This is not unexpected as the flow can separate either from the upstream or downstream part of the 2 ft thick wall between the head-breaker chamber and the surge chamber. As a result, a revision to the design was made by the engineer which incorporated a rounding of the upstream edges of the gate openings. This change is expected to increase the discharge coefficient of the gates, such that

[1]Henderson, F. M., Open Channel Flow, MacMillan Company, N. Y. 1966.

$$Q_{actual} = 1.2\ Q_{measured}$$

per the Engineer. It was decided, however, not to change the model for purposes of documentation.

C. Transition Chamber

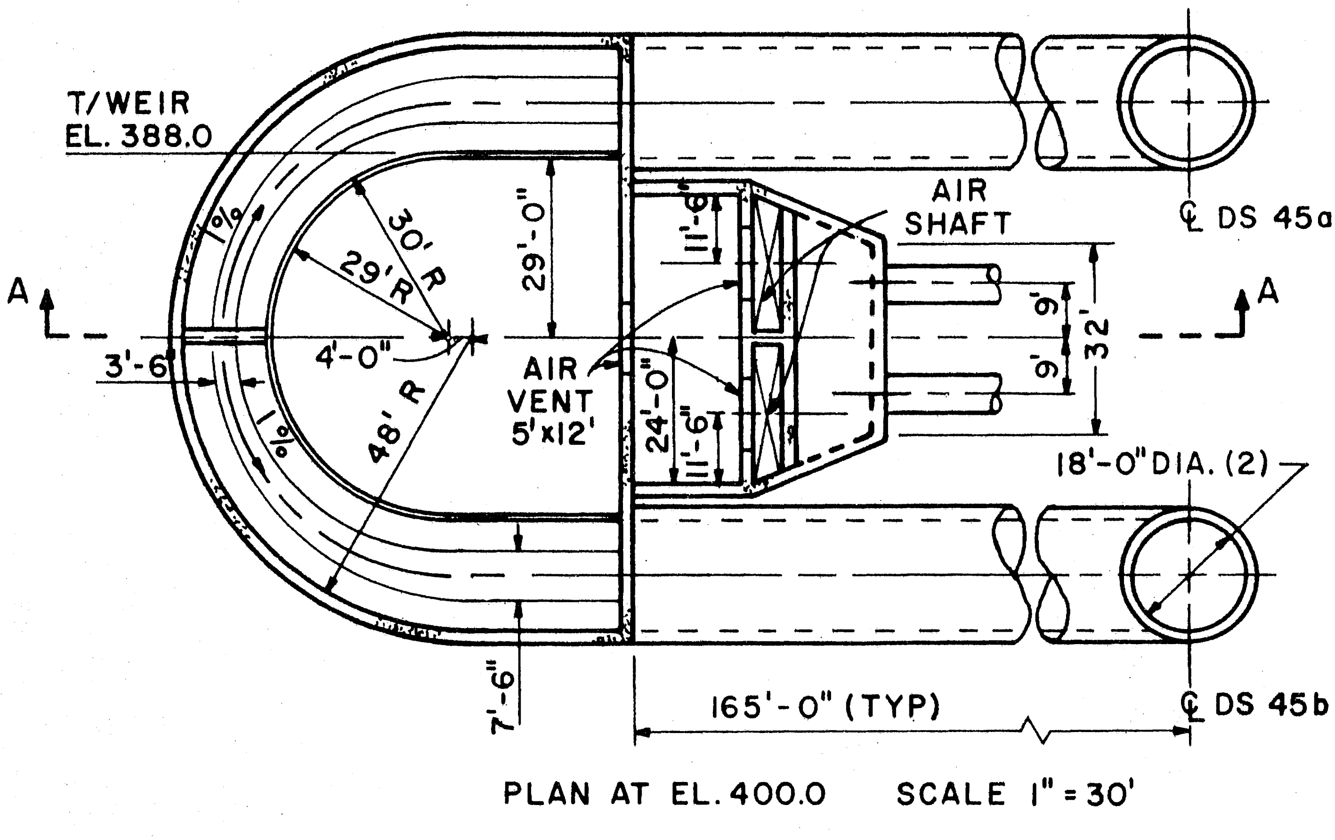
Removal of air entrained in the flow by jets issuing from the head-breaker gates occurs in the transition chamber. The head-discharge relationship of the 6 ft conduits determines the water level in the transition and head-breaker chambers. A range of conditions from free flow to submerged flow of the highest gates on the head-breaker wall were examined. Air removal was satisfactory.

D. Collection Chamber

Figure 18 shows the water surface profile along the centerline of the circumferential collection channel and overflow conduit. The flow is 1500 cfs. A dotted line shows the calculated critical depth for an 18 ft diameter circular section. For uniform subcritical flow, a transition to supercritical flow can only occur at a break in grade such as the brink; however, accelerated flow before the brink allows transition to occur a very short distance upstream.

The slope of the channel surrounding the surge chamber is 1 percent which is a steep slope, and supercritical flow would occur except for the retarding effect of the side flow from the weir. The normal depth in the side channel is greater than the expected normal depth in the 18 ft conduits. This produces a hydraulic drop at the wall between the side channel and the 18 ft diameter conduits followed by a standing wave. This can be noted immediately downstream of the wall on Fig. 18.

Channel roughness was not modeled in the 18 ft conduits; instead the slope was adjusted to give a normal depth between the critical depth and the center line. A drawdown of the water surface profile occurs for approximately 50 ft upstream of the point of discharge into the dropshafts.



PERTINENT DIMENSIONS OF THE SITE 45 STRUCTURE

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UNIVERSITY OF MINNESOTA

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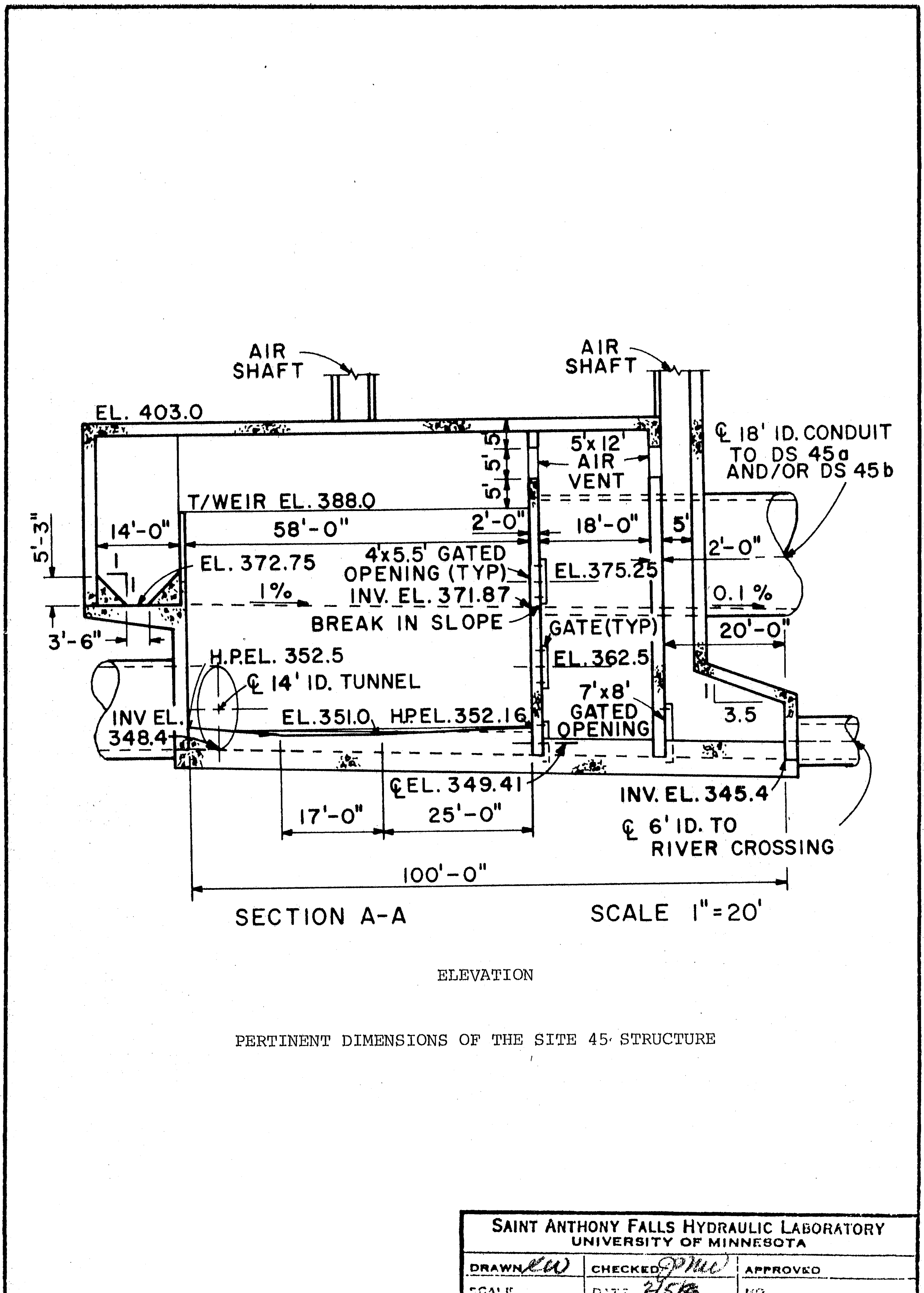
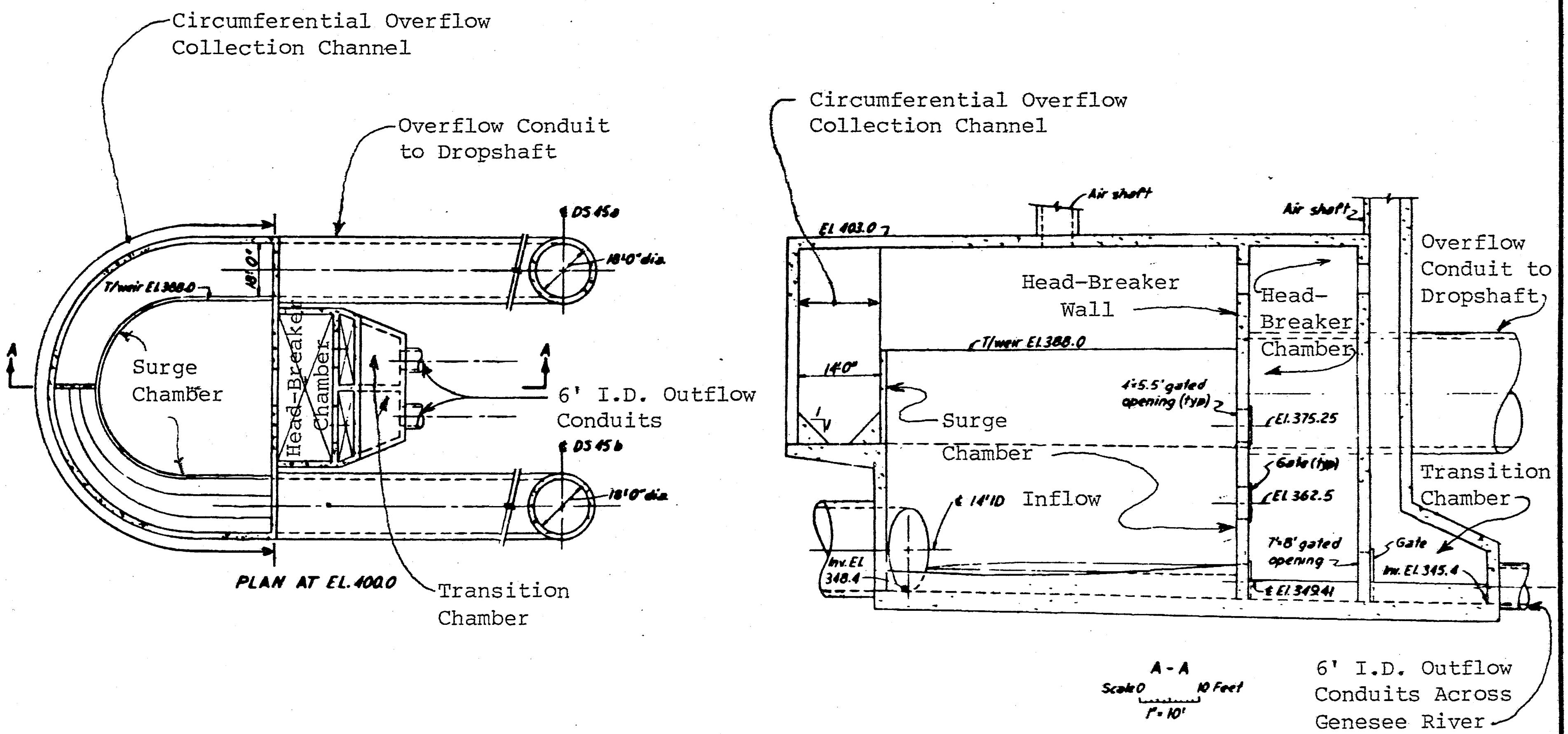
SCALE	DATE	NO.



ELEVATION

PERTINENT DIMENSIONS OF THE SITE 45 STRUCTURE



SIMPLIFIED SKETCH OF THE SITE 45 STRUCTURE

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UNIVERSITY OF MINNESOTA

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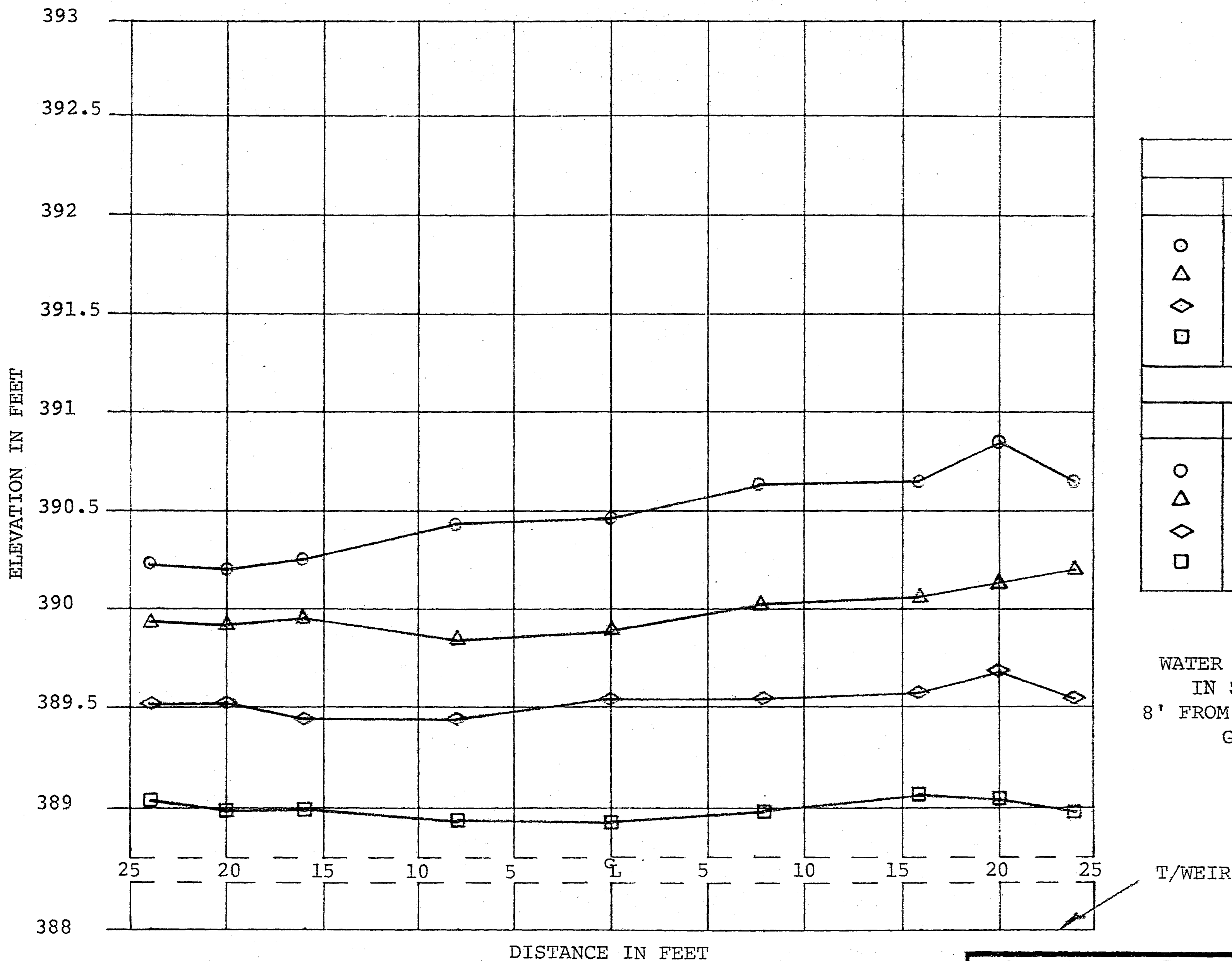
SCALE	DATE 2/5/85	NO.

FIGURE 2



	INFLOW	
	Q_L	Q_R
○	2,000	0
△	1,500	0
◇	1,000	0
□	500	0

	OUTFLOW	
	Q_L	Q_R
○	1,050	950
△	750	750
◇	500	500
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
8' FROM HEAD BREAKER WALL
GATES CLOSED

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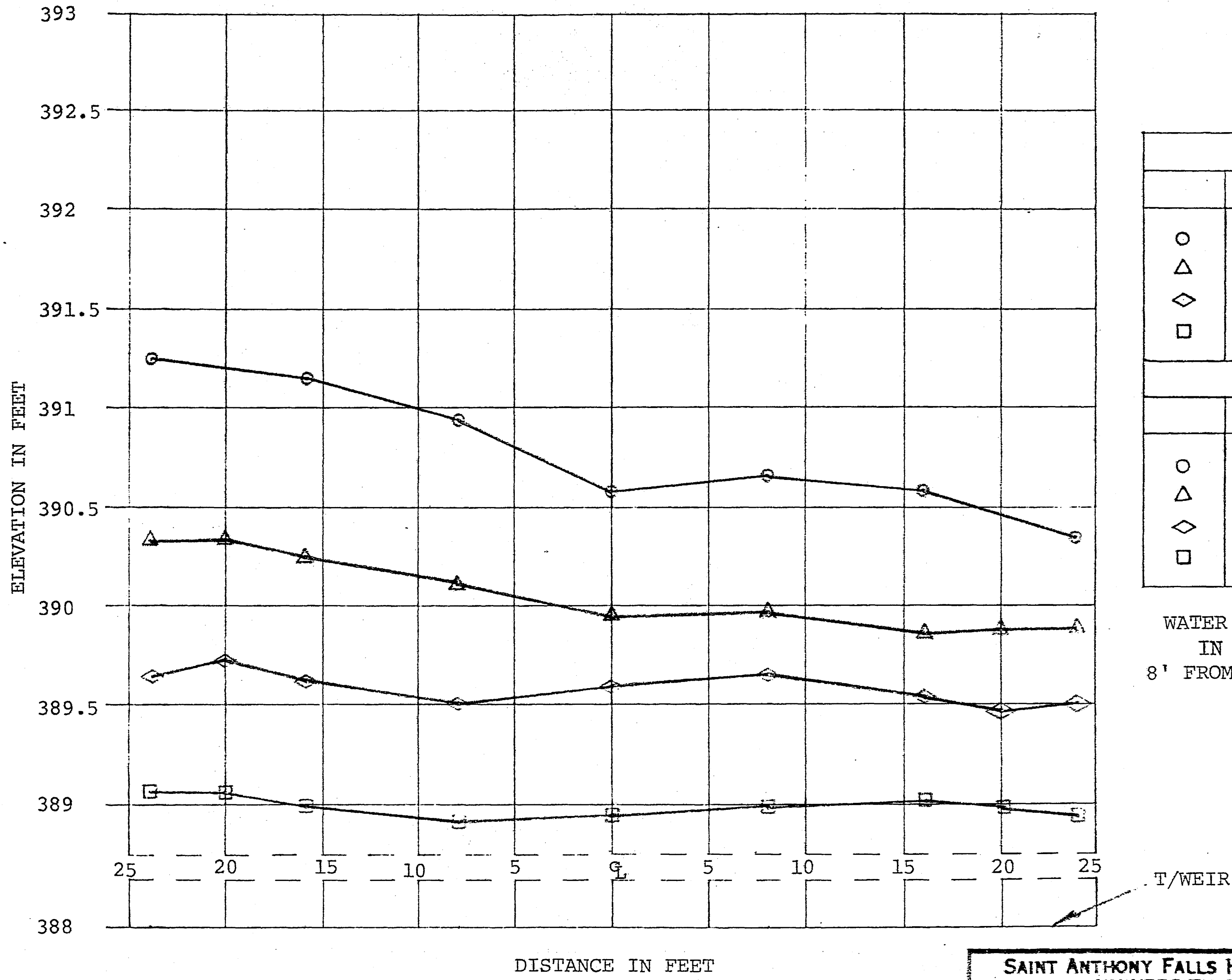
FIGURE 3



INFLOW		
	Q_L	Q_R
○	0	2,000
△	0	1,500
◇	0	1,000
□	0	500

OUTFLOW		
	Q_L	Q_R
○	1,000	1,000
△	750	750
◇	470	530
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
8' FROM HEAD BREAKER WALL
GATES CLOSED

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
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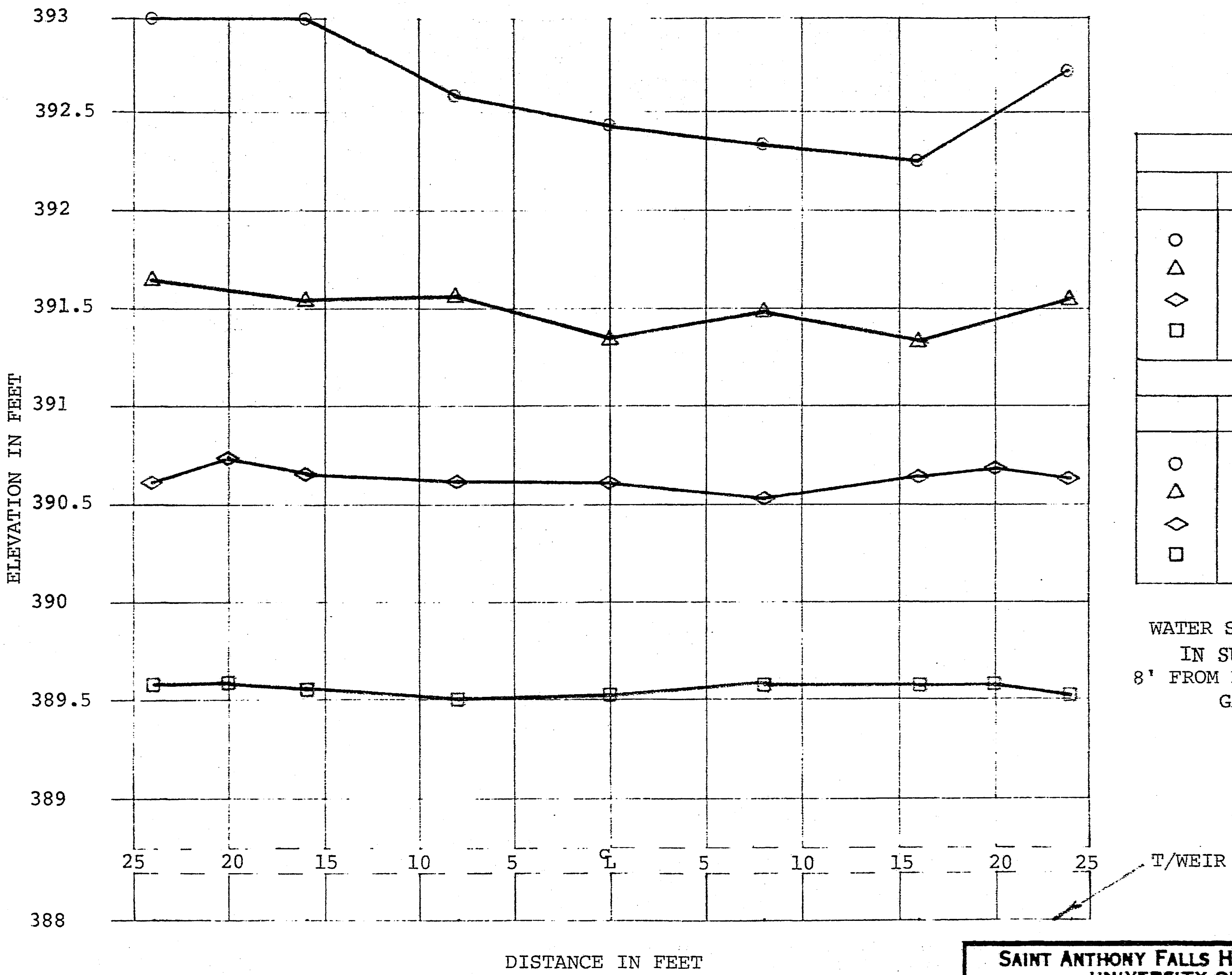
FIGURE 4



INFLOW		
	Q_L	Q_R
○	2,000	2,000
△	1,500	1,500
◇	1,000	1,000
□	500	500

OUTFLOW		
	Q_L	Q_R
○	2,000	2,000
△	1,650	1,350
◇	1,025	975
□	500	500

WATER SURFACE PROFILES
IN SURGE CHAMBER
8' FROM HEAD BREAKER WALL
GATES CLOSED

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
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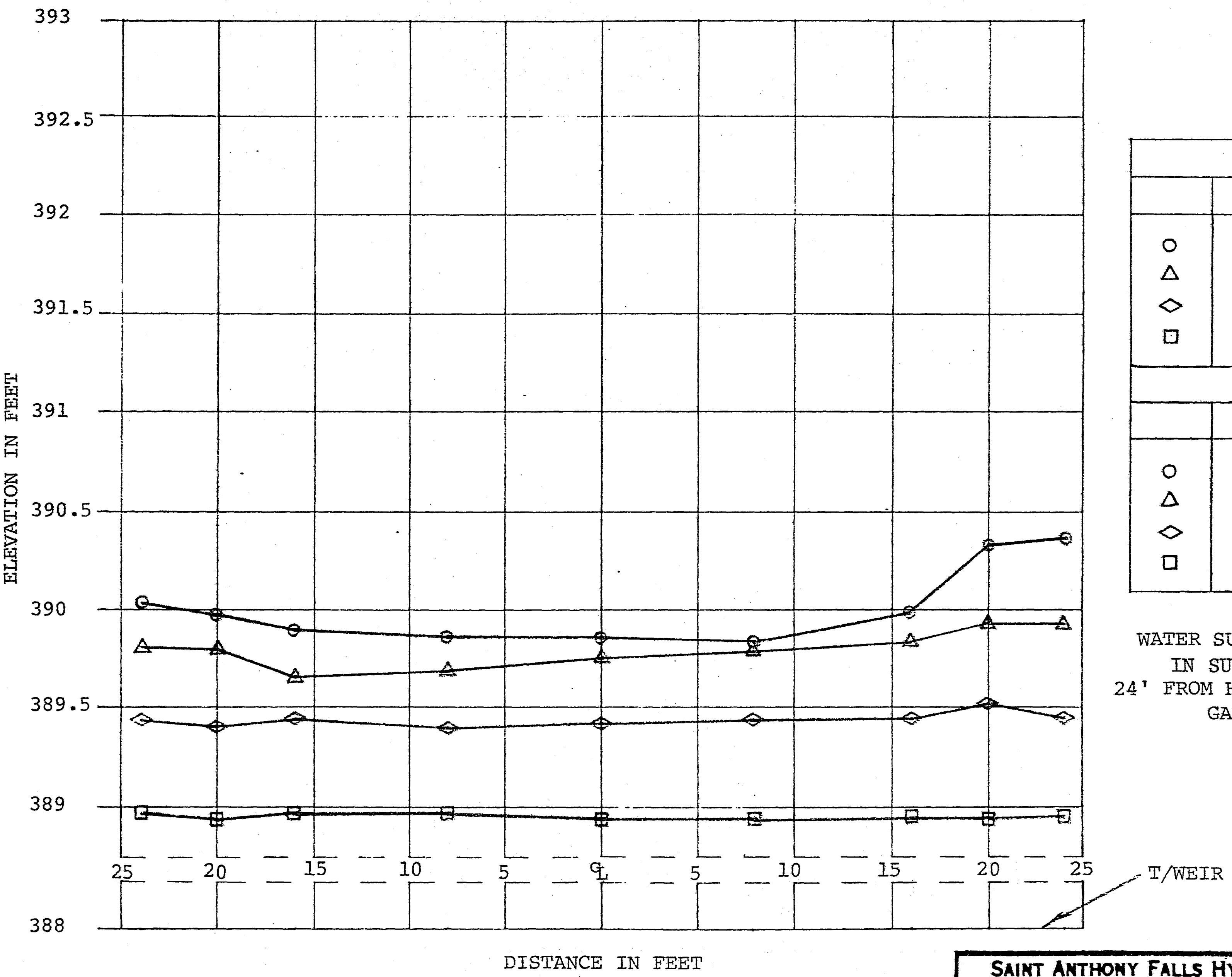
FIGURE 5



INFLOW		
	Q_L	Q_R
○	2,000	0
△	1,500	0
◇	1,000	0
□	500	0

OUTFLOW		
	Q_L	Q_R
○	1,050	950
△	750	750
◇	500	500
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
24' FROM HEAD BREAKER WALL
GATES CLOSED

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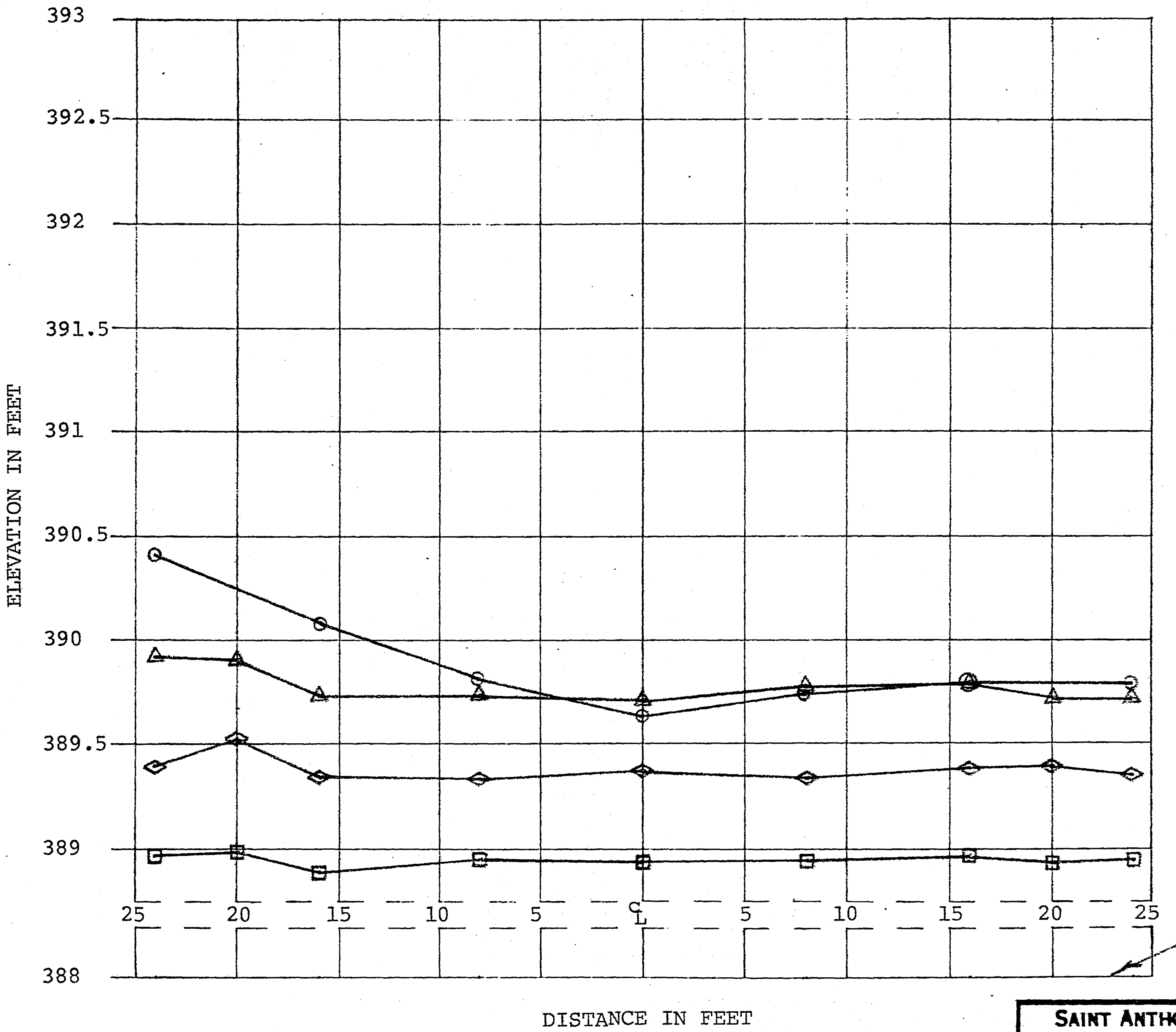
FIGURE 6



INFLOW		
	Q_L	Q_R
○	0	2,000
△	0	1,500
◇	0	1,000
□	0	500

OUTFLOW		
	Q_L	Q_R
○	1,000	1,000
△	750	750
◇	480	520
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
24' FROM HEAD BREAKER WALL
GATES CLOSED

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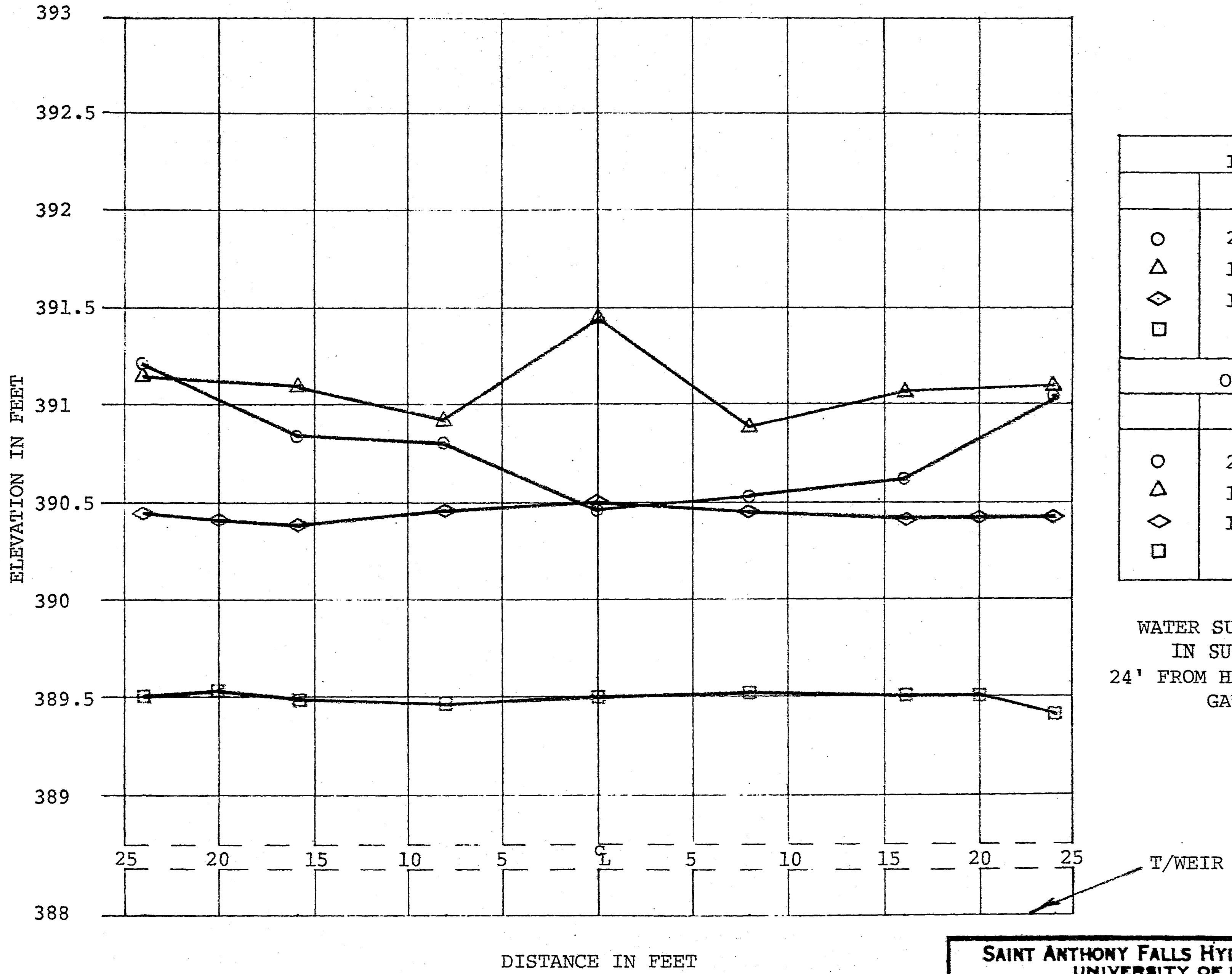
FIGURE 7



	INFLOW	
	Q_L	Q_R
○	2,000	2,000
△	1,500	1,500
◇	1,000	1,000
□	500	500
	OUTFLOW	
	Q_L	Q_R
○	2,000	2,000
△	1,650	1,350
◇	1,025	975
□	500	500

WATER SURFACE PROFILES
IN SURGE CHAMBER
24' FROM HEAD BREAKER WALL
GATES CLOSED

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UNIVERSITY OF MINNESOTA

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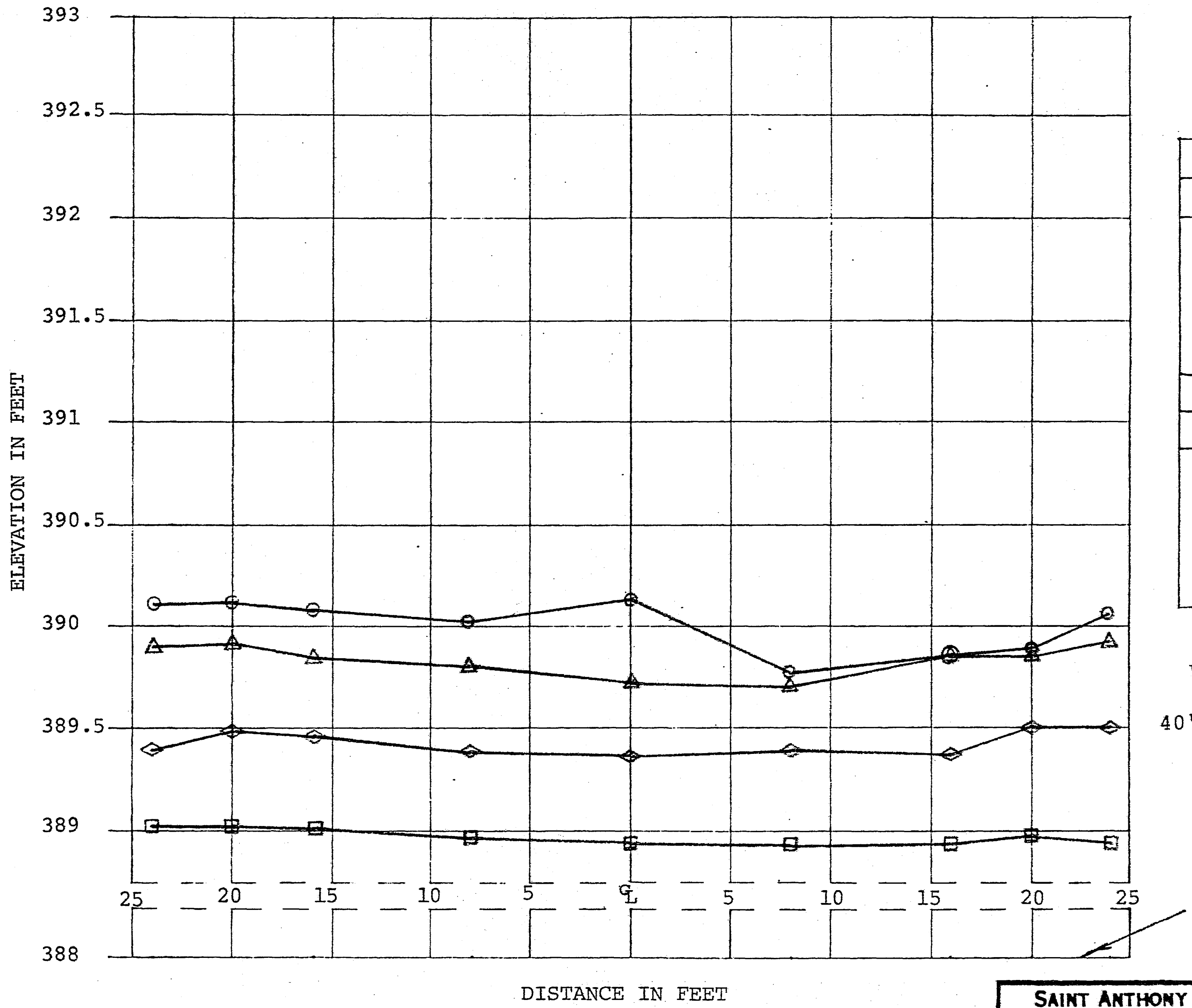
FIGURE 8



INFLOW		
	Q_L	Q_R
○	2,000	0
△	1,500	0
◇	1,000	0
□	500	0

OUTFLOW		
	Q_L	Q_R
○	1,050	950
△	750	750
◇	500	500
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
40' FROM HEAD BREAKER WALL
GATES CLOSED

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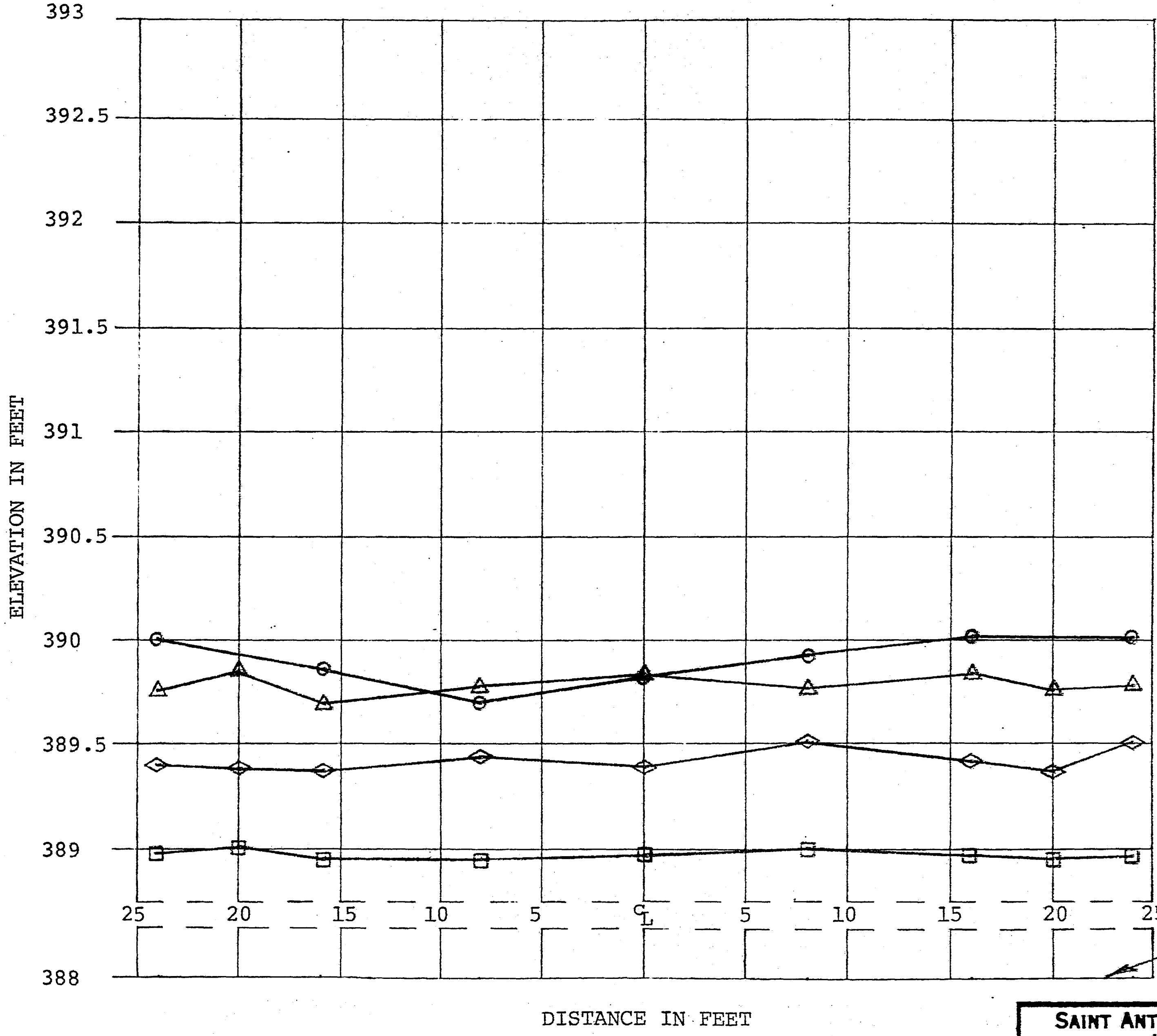
FIGURE 9



INFLOW		
	Q_L	Q_R
○	0	2,000
△	0	1,500
◇	0	1,000
□	0	500
OUTFLOW		
	Q_L	Q_R
○	1,000	1,000
△	750	750
◇	470	530
□	250	250

WATER SURFACE PROFILES
IN SURGE CHAMBER
40' FROM HEAD BREAKER WALL
GATES CLOSED

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

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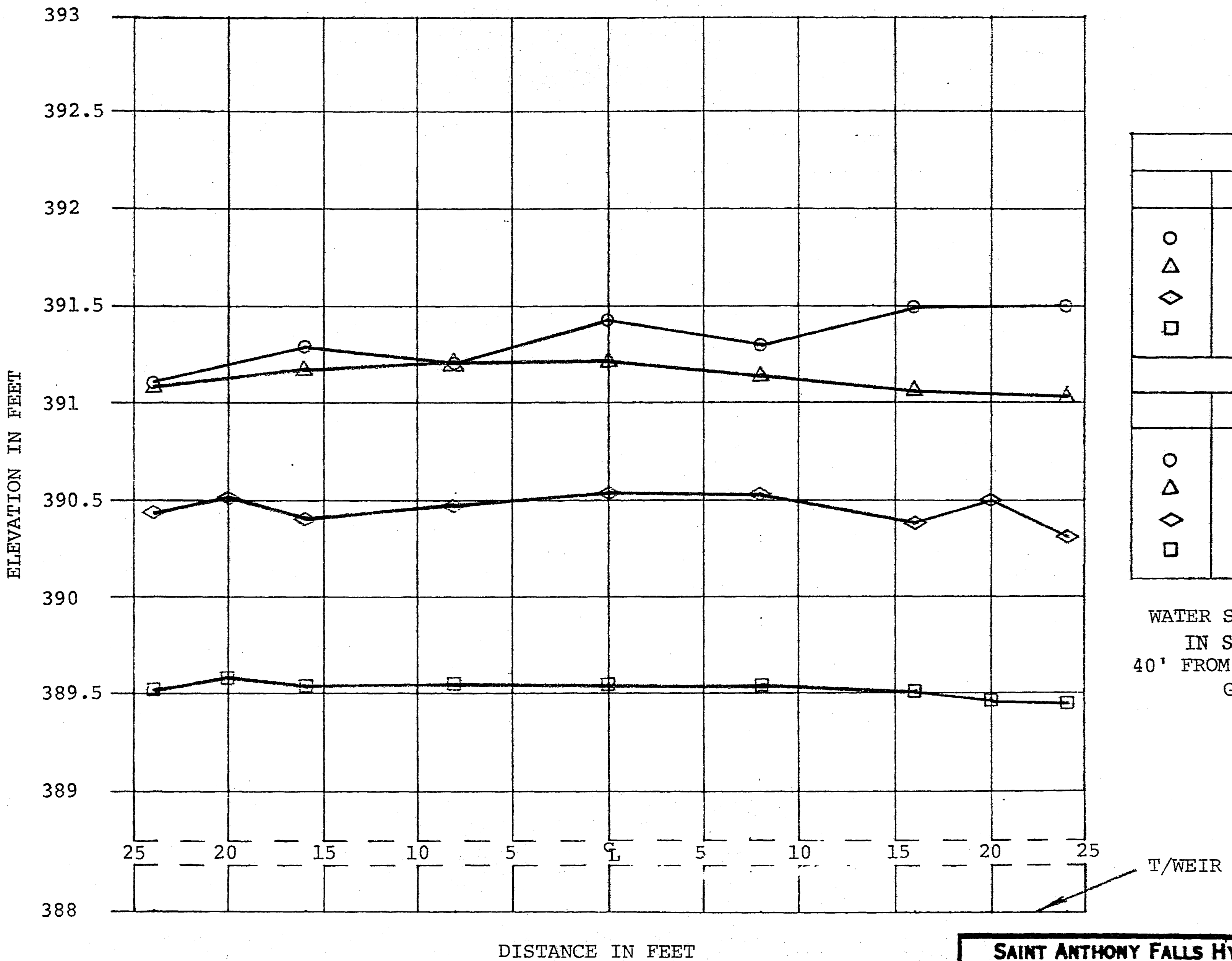
FIGURE 10



INFLOW		
	Q_L	Q_R
○	2,000	2,000
△	1,500	1,500
◇	1,000	1,000
□	500	500
OUTFLOW		
	Q_L	Q_R
○	1,900	2,100
△	1,650	1,350
◇	1,025	975
□	500	500

WATER SURFACE PROFILES
IN SURGE CHAMBER
40' FROM HEAD BREAKER WALL
GATES CLOSED

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
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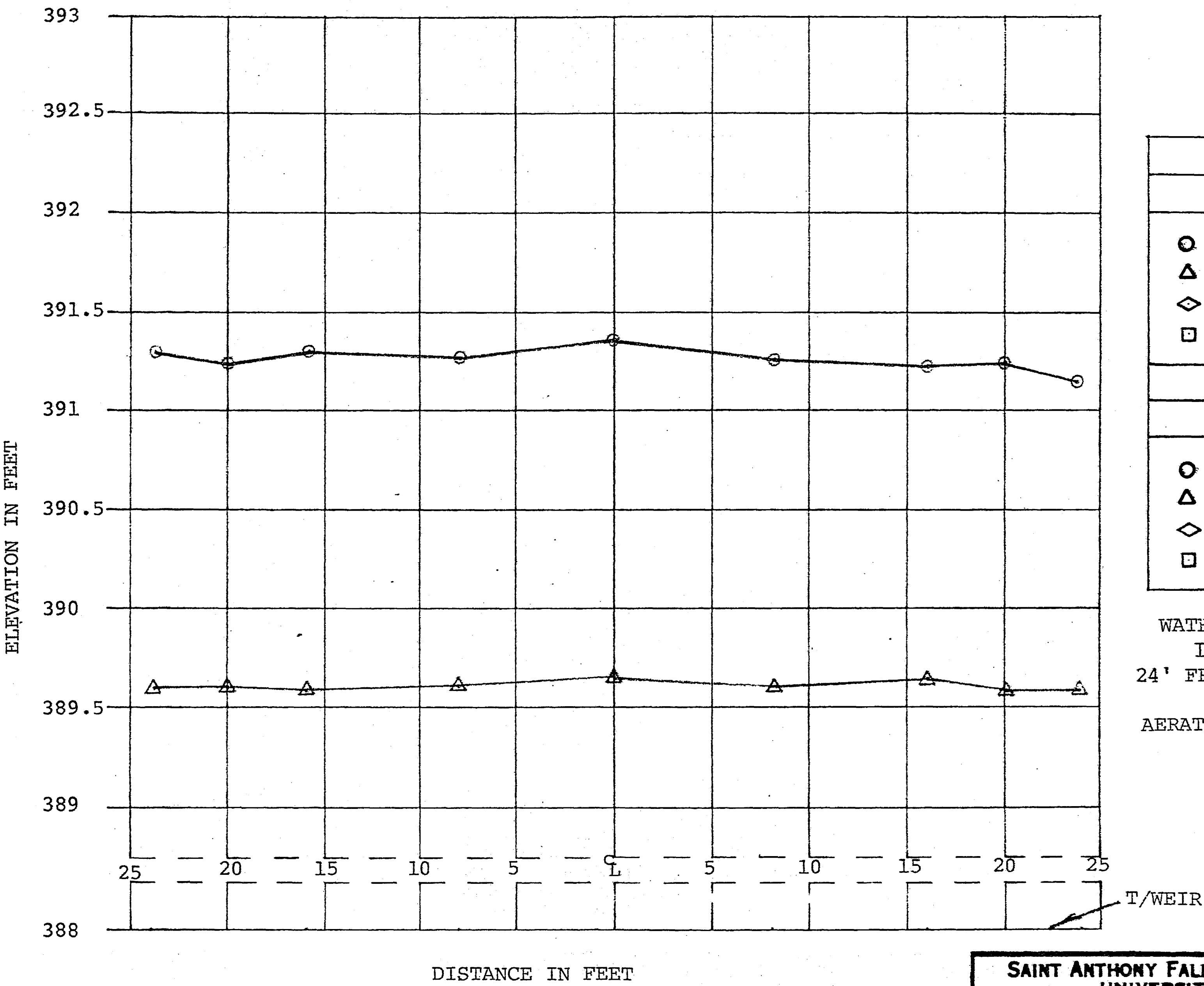
SCALE	DATE *3/11/52*	NO.

FIGURE 11



INFLOW		
	Q_L	Q_R
○	1,500	1,500
△	500	500
◇	——	——
⊡	——	——
OUTFLOW		
	Q_L	Q_R
○	1,500	1,500
△	500	500
◇	——	——
⊡	——	——

WATER SURFACE PROFILES
IN SURGE CHAMBER
24' FROM HEAD BREAKER WALL
GATES CLOSED
AERATION BLOCKS INSTALLED

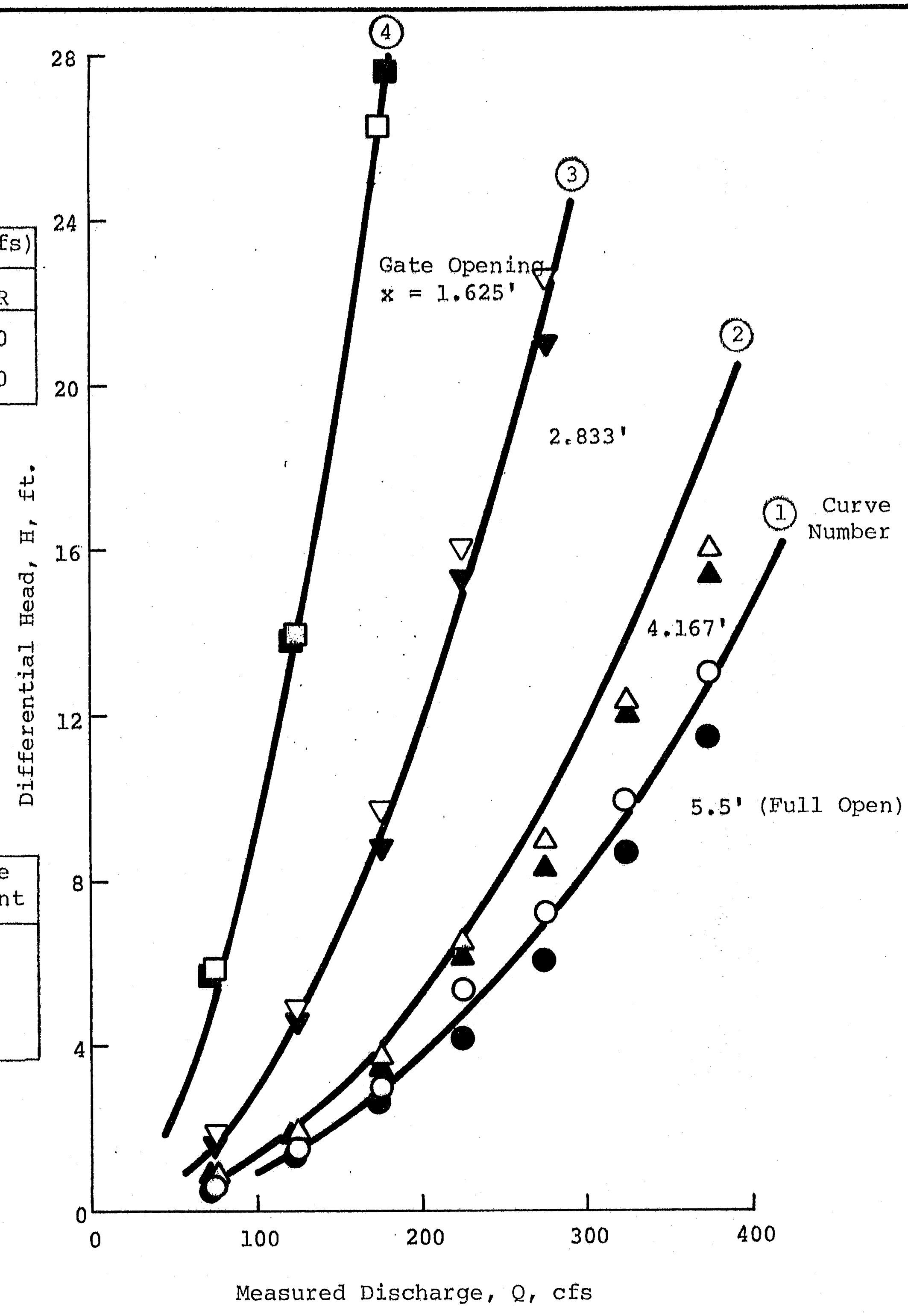
SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN TO	CHECKED GF	APPROVED
SCALE	DATE 7/30/82	NO.

FIGURE 12

LEGEND

Symbol	Inflow (cfs)	
	Q_L	Q_R
Open	250	250
Filled	1500	1500

Curve Number	Discharge Coefficient
1	0.59
2	0.65
3	0.65
4	0.65



MEASURED HEAD-DISCHARGE CURVES, LOWER GATE

NOTE:

Upstream corners will be rounded in the prototype so that $Q_{actual} = 1.2\ Q_{measured}$, per the Engineer.

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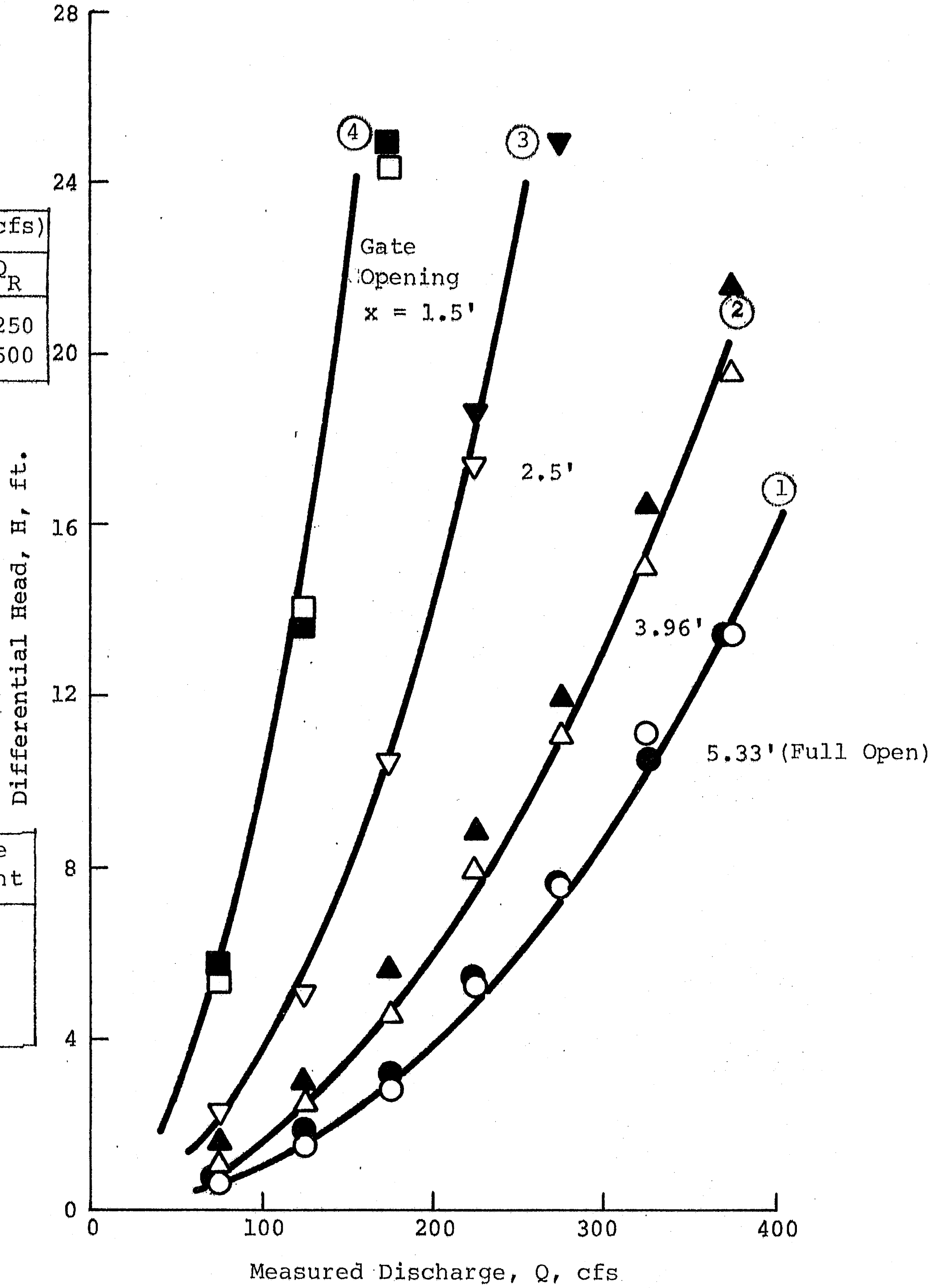
DRAWN	CHECKED	APPROVED
SCALE	DATE 1/4/84	NO.

LEGEND

Symbol	Inflow (cfs)	
	Q_L	Q_R
Open	250	250
Filled	1500	1500

Curve Number	Discharge Coefficient
1	0.59
2	0.65
3	0.65
4	0.65



MEASURED HEAD-DISCHARGE CURVES, MIDDLE GATE

NOTE:

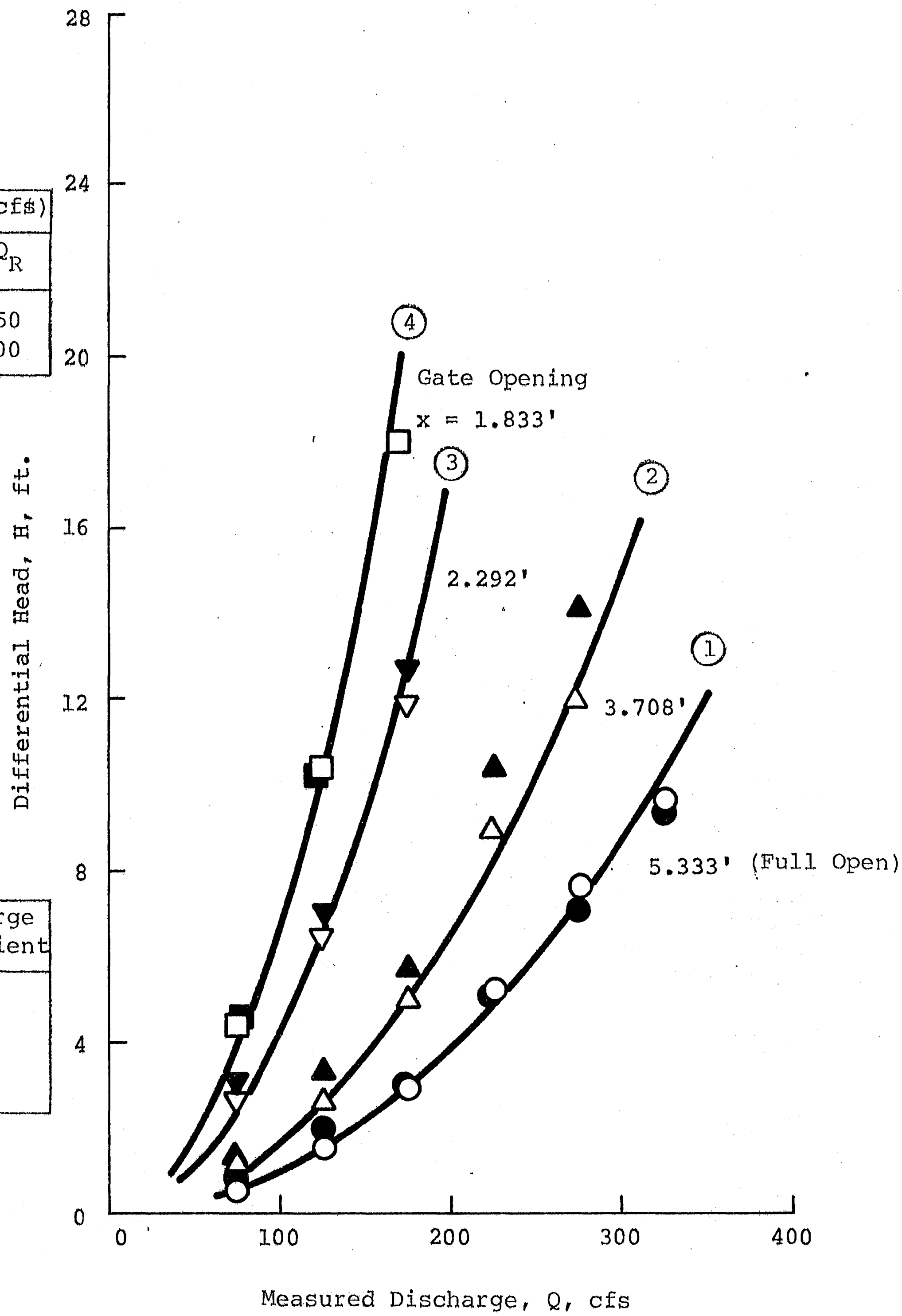
Upstream corners will be rounded in the prototype so that Q_{actual} = 1.2 $Q_{measured}$ per the Engineer.

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN	CHECKED	APPROVED
SCALE	DATE	NO.

LEGEND

Symbol	Inflow (cfs)	
	Q_L	Q_R
Open	250	250
Filled	1500	1500

Curve Number	Discharge Coefficient
1	0.59
2	0.65
3	0.65
4	0.65



MEASURED HEAD-DISCHARGE CURVES, TOP GATE

NOTE:

Upstream corners will be rounded in the prototype so that Q_{actual} = 1.2 $Q_{measured}$ per the Engineer.

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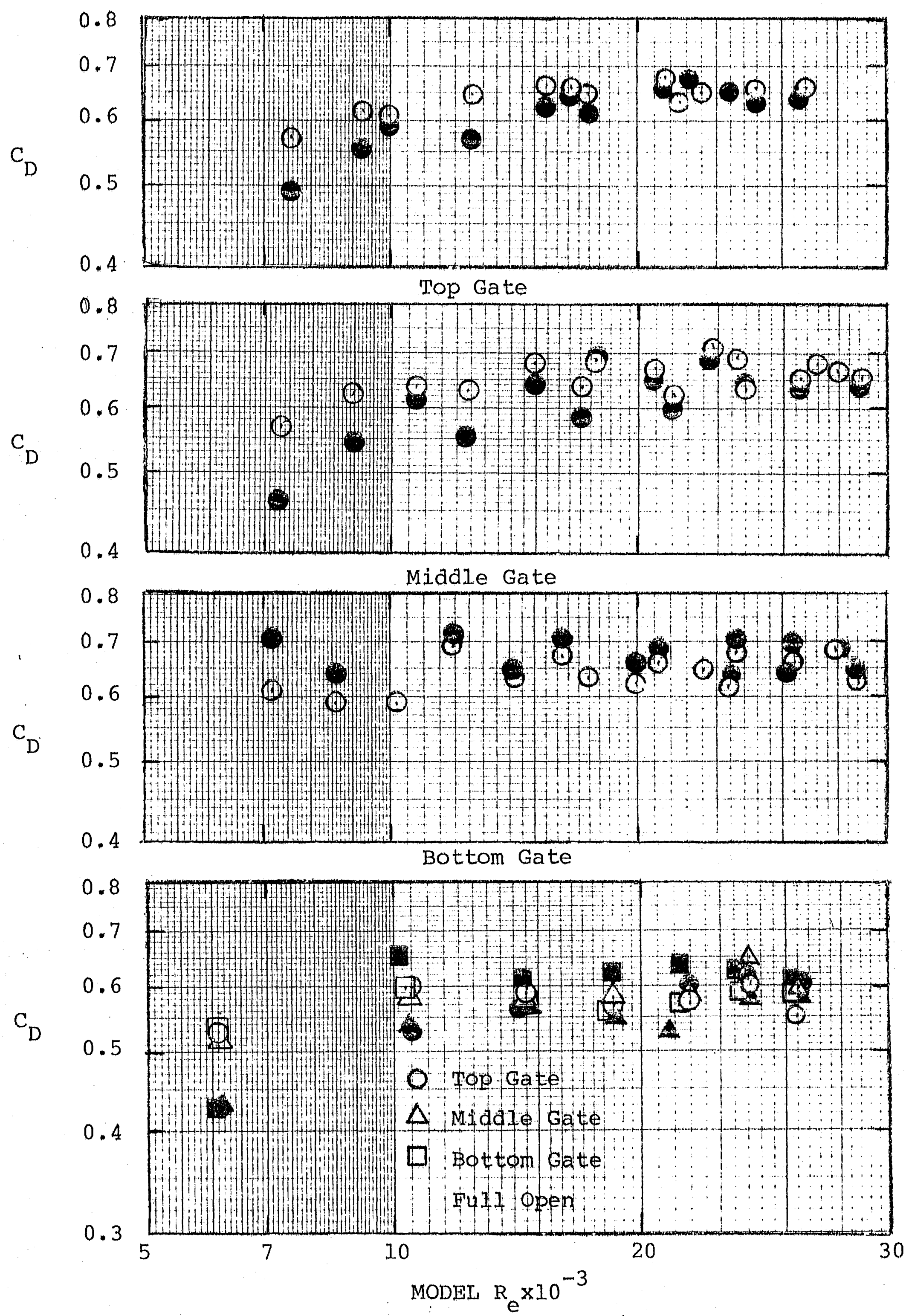
DRAWN	CHECKED	APPROVED
SCALE	DATE 11/5/81	NO.

FIGURE 16



VARIATION OF DISCHARGE COEFFICIENT WITH REYNOLDS NUMBER

Symbols	Q_T, cfs
Open	500
Filled	3000

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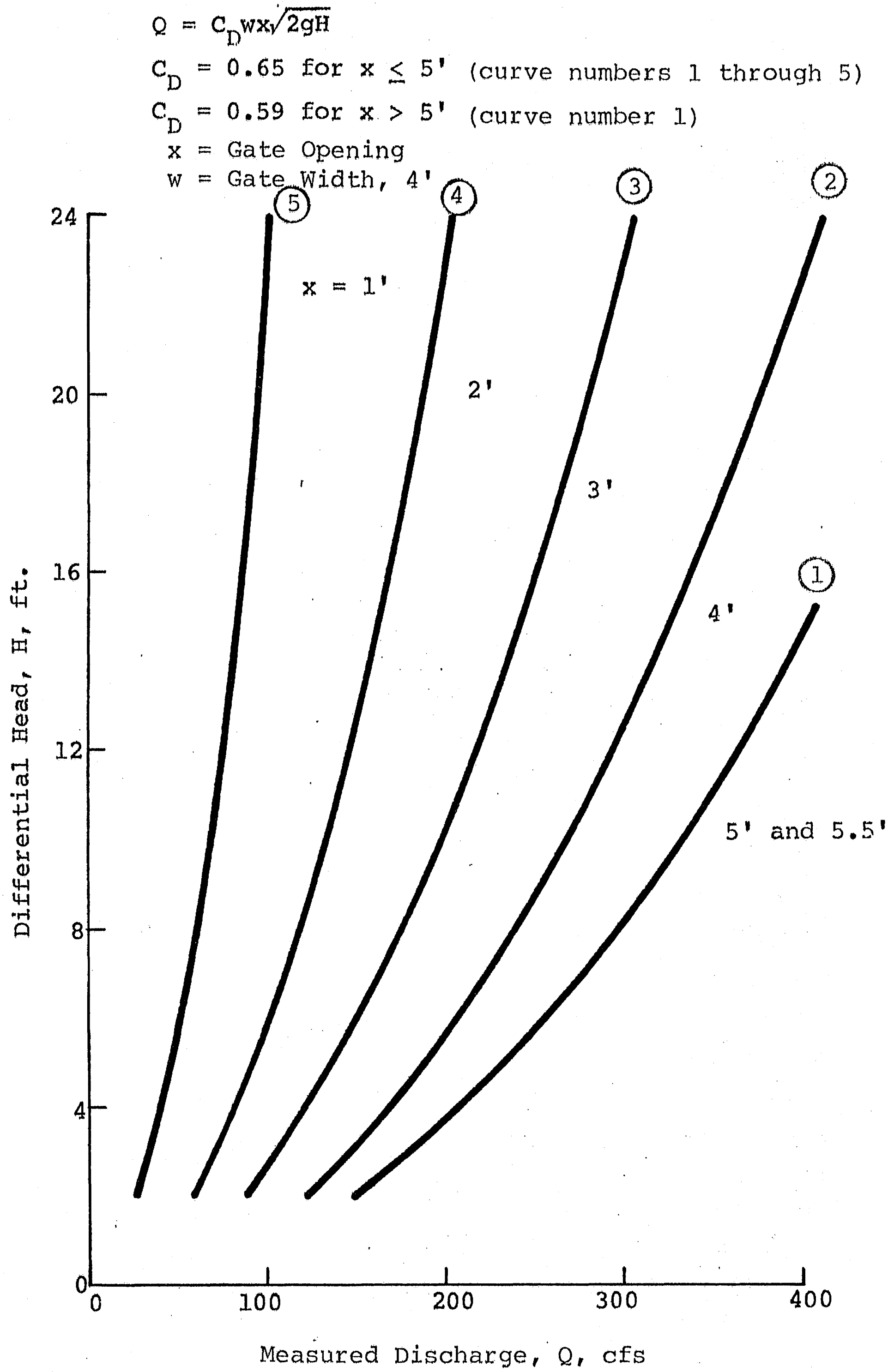
DRAWN JA	CHECKED MW	APPROVED
SCALE	DATE 10/20/84	NO.

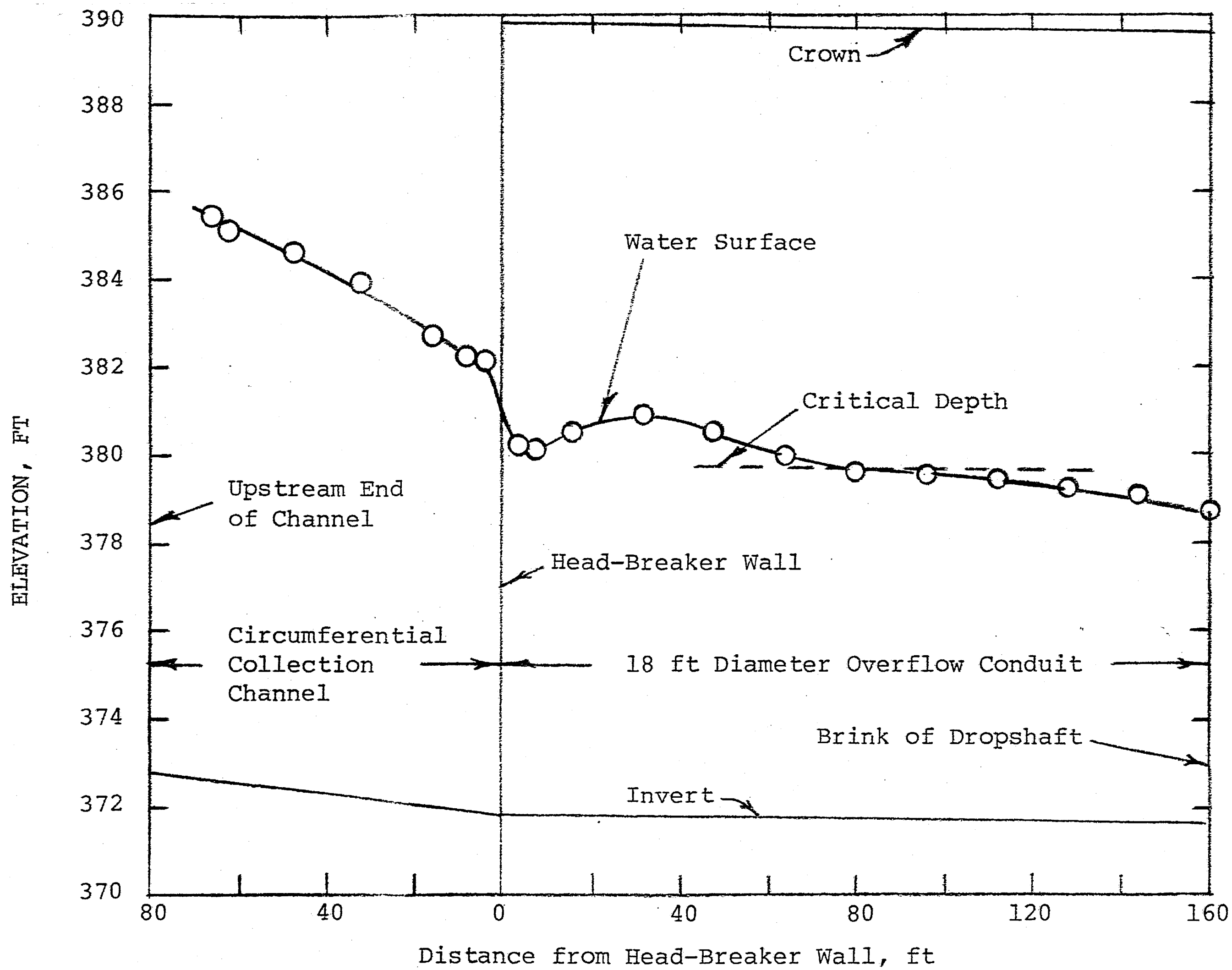


AVERAGED HEAD-DISCHARGE CURVES, ALL GATES , SHARP-EDGED OPENINGS

NOTE:

Upstream corners will be rounded in the prototype so that $Q_{actual} = 1.2\ Q_{measured}$ per the Engineer.

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN RB	CHECKED JMW	APPROVED JW
SCALE	DATE 11/5/79	NO.



WATER SURFACE PROFILE ALONG CENTERLINE OF CIRCUMFERENTIAL COLLECTION CHANNEL AND OVERFLOW CONDUIT, Q = 1500 cfs

FIGURE 18

SAINT ANTHONY FALLS HYDRAULIC LABORATORY UNIVERSITY OF MINNESOTA		
DRAWN JCW	CHECKED JMW	APPROVED
SCALE	DATE 10/21/84	NO.

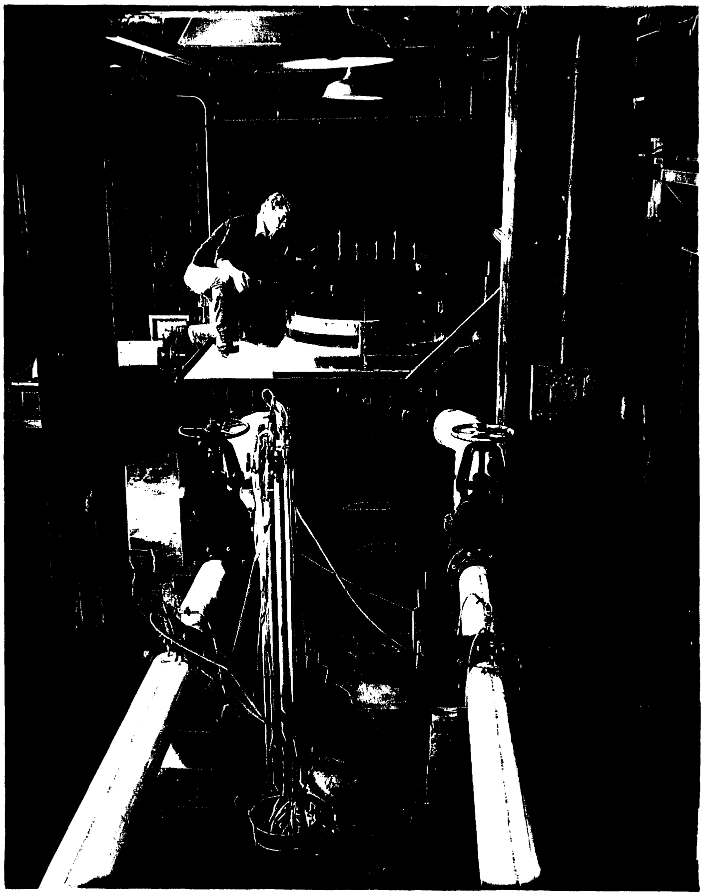


Photo 1. View from upstream of the model.

Photo 2. Downstream view of model.

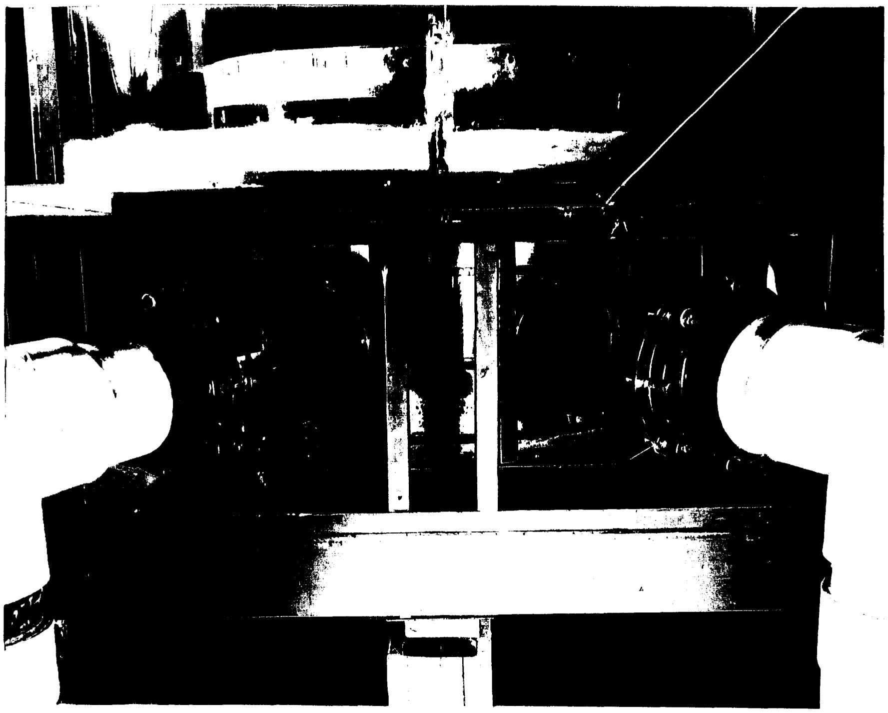


Photo 3. 14 ft I.D. conduit entering surges riser.

Photo No. 4. Weir, surge chamber, and circumferential collection channel.

Photo No. 5. Weir and circumferential collection channel with point gage carriage.

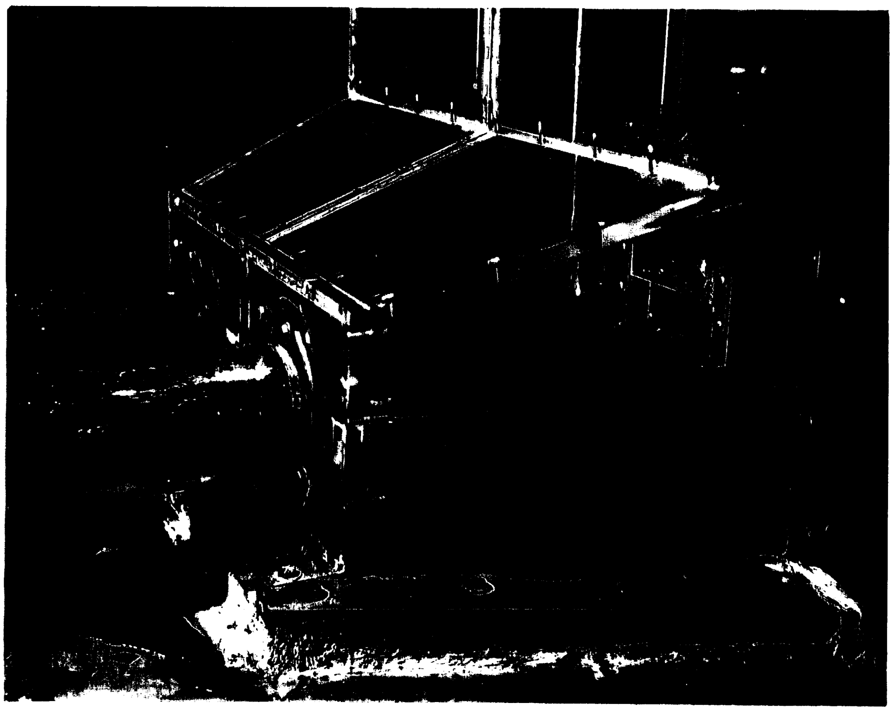


Photo 6. Transition chamber.

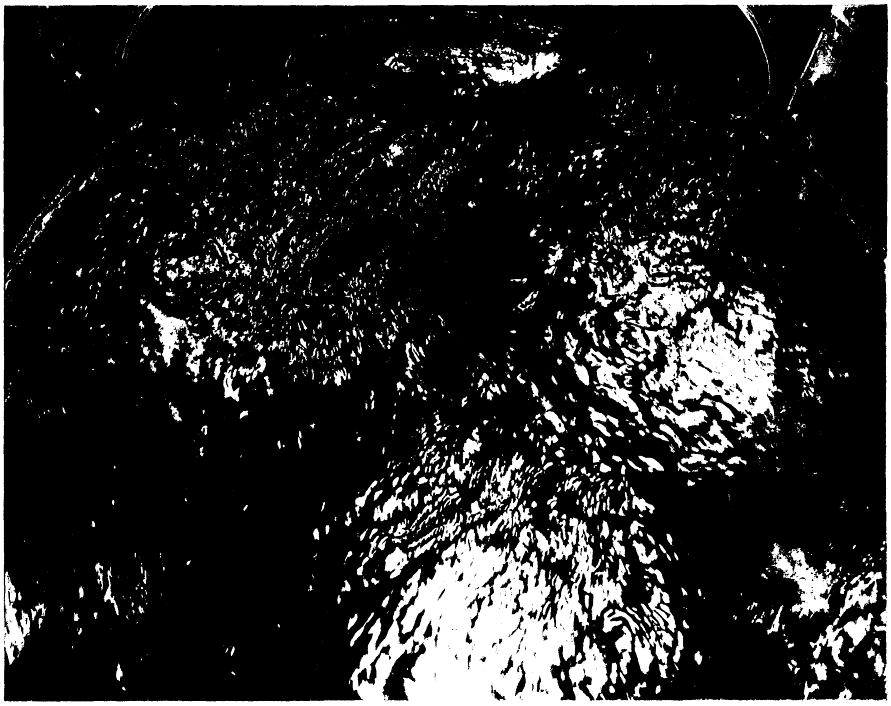


Photo 7. Flow in surge chamber, $Q_R = Q_L = 187.5$ cfs.

Photo 8. Flow in surge chamber, $Q_R = Q_L = 75$ cfs.

Photo 9. Surge chamber draining.

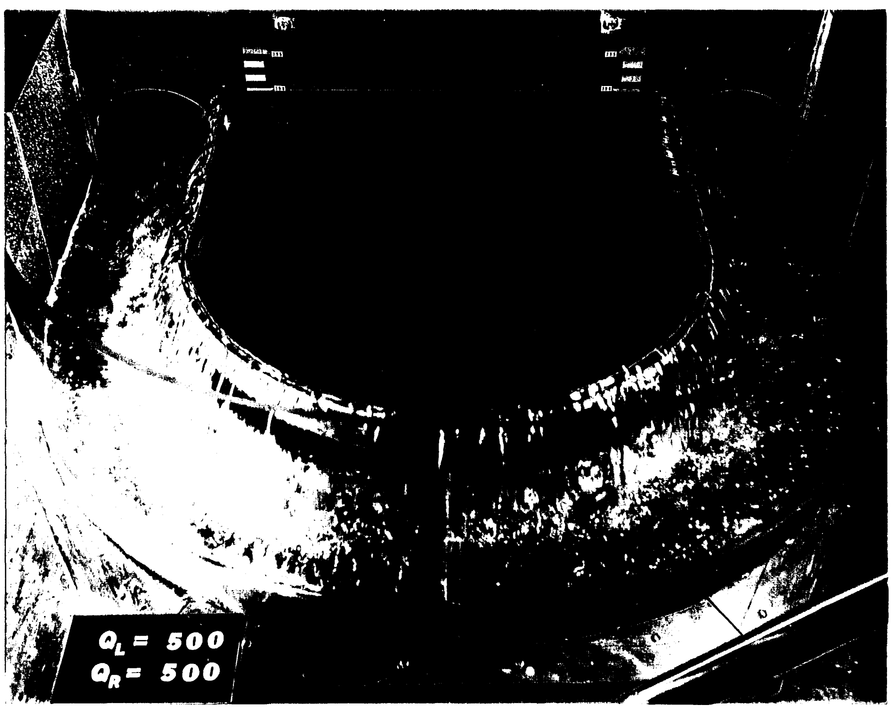


Photo 10. Flow over relief weir, Inflow $Q_R = Q_L = 500$; Outflow $Q_R = 500$, $Q_L = 500$ cfs.

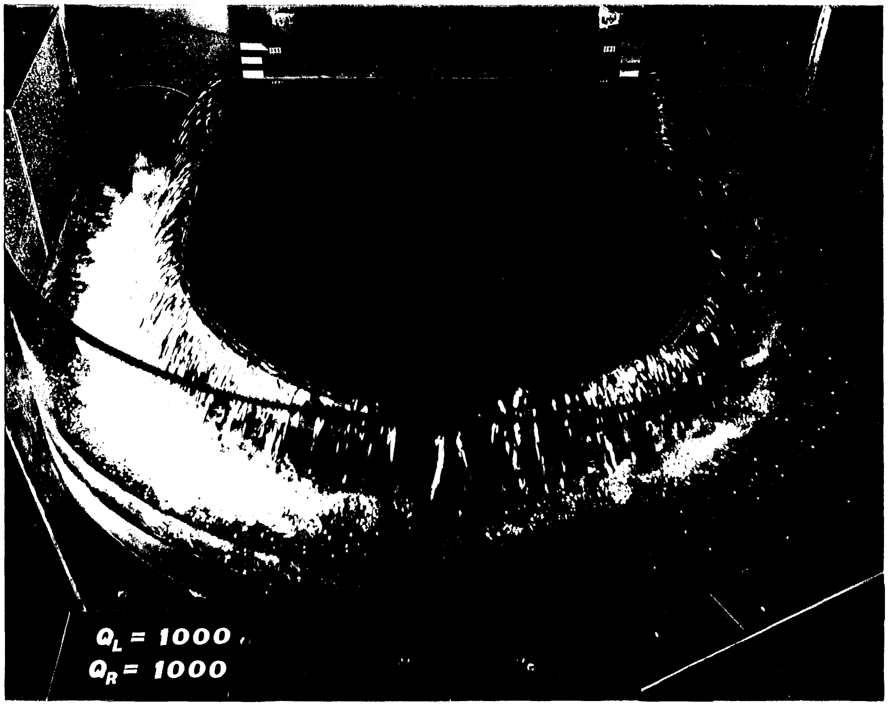


Photo 11. Flow over relief weir, Inflow Q_R = Q_L = 1000; Outflow Q_R = 975, Q_L = 1025 cfs.

Photo 12. Flow over relief weir, Inflow Q_R = Q_L = 1450; Outflow Q_R = 1250, Q_L = 1650 cfs.

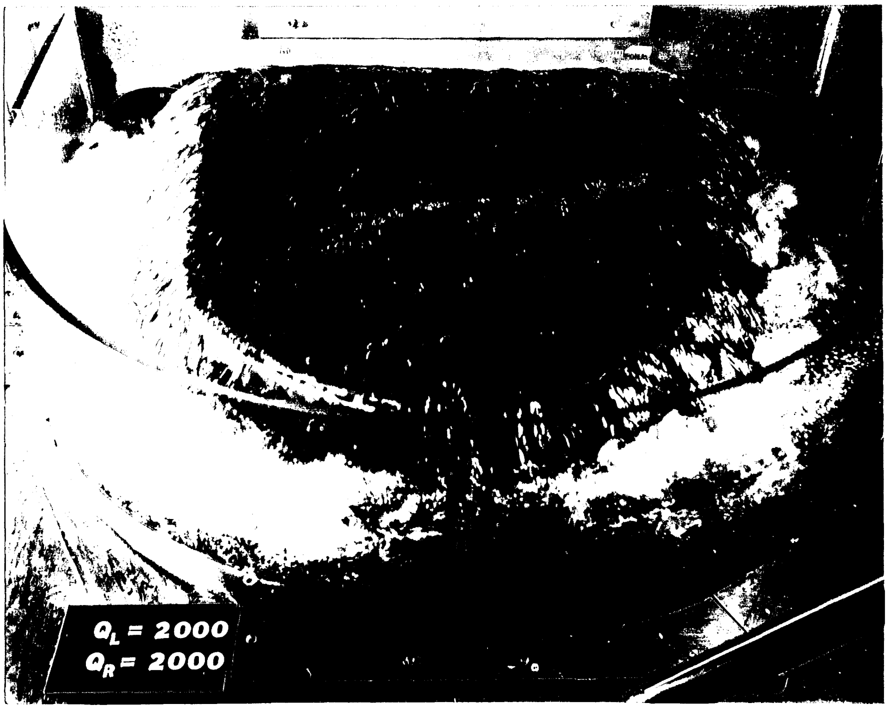


Photo 13. Flow over relief weir, Inflow Q_R = Q_L = 2000; Outflow Q_R = 2000, Q_L = 2000 cfs.

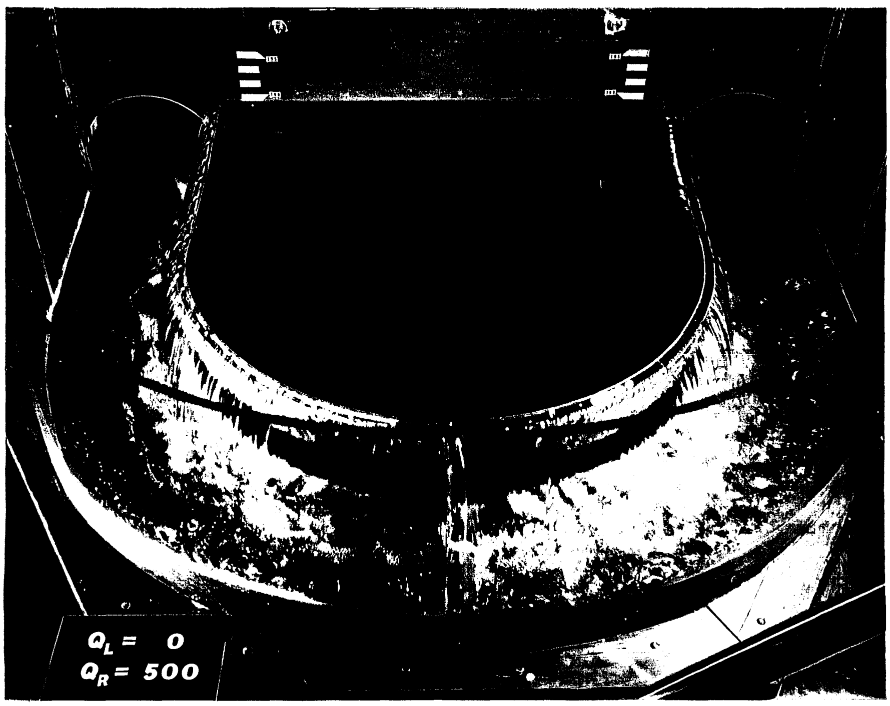


Photo 14. Flow over relief weir, Inflow Q_R = 500, Q_L = 0; Outflow Q_R = 250, Q_L = 250 cfs.

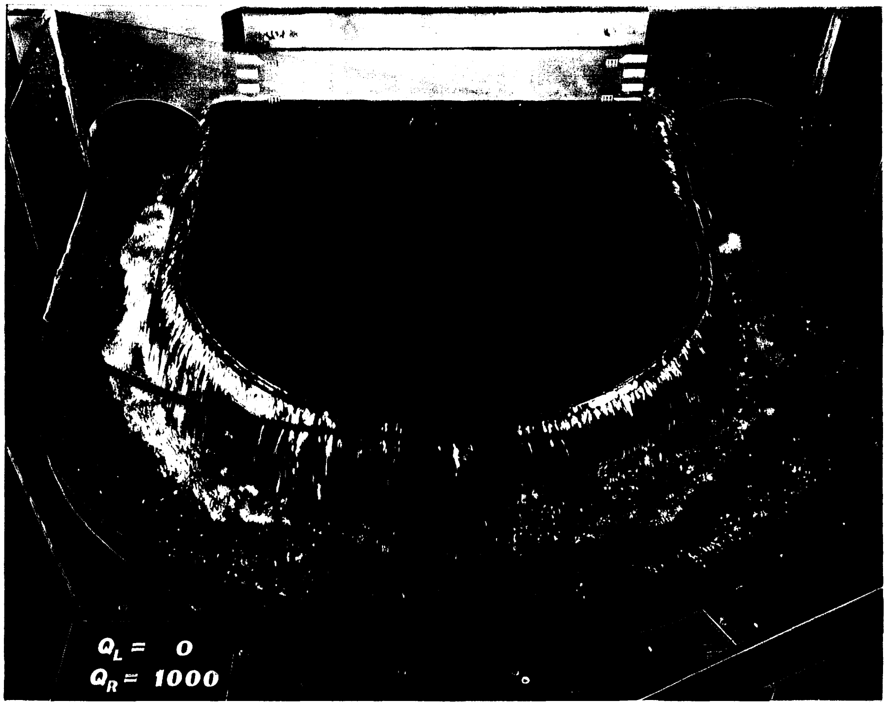


Photo 15. Flow over relief weir, Inflow Q_R = 1000, Q_L = 0; Outflow Q_R = 530, Q_L = 470 cfs.

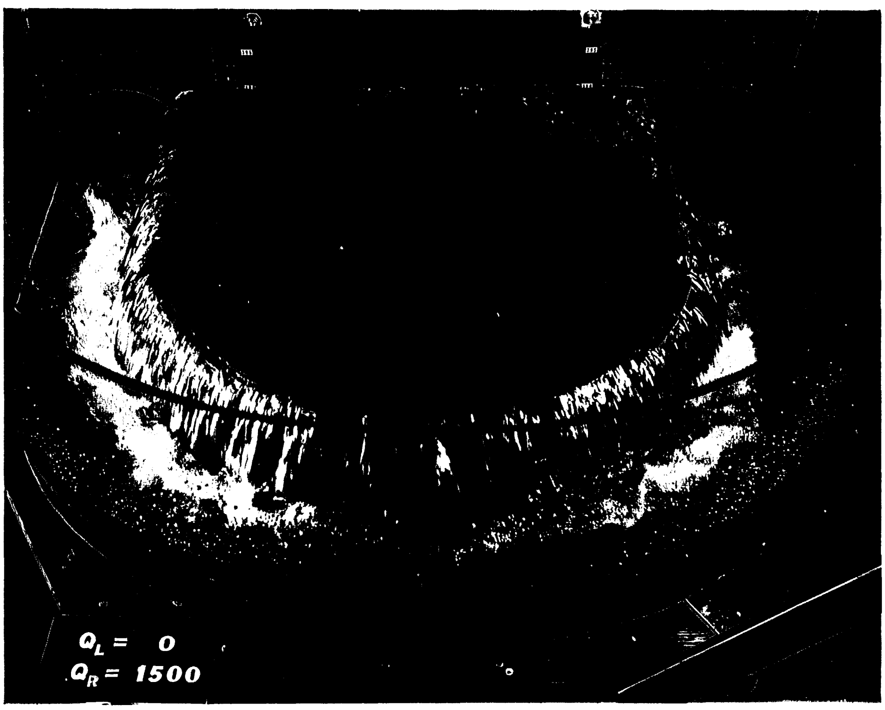


Photo 16. Flow over relief weir, Inflow Q_R = 1500, Q_L = 0; Outflow Q_R = 750, Q_L = 750 cfs.

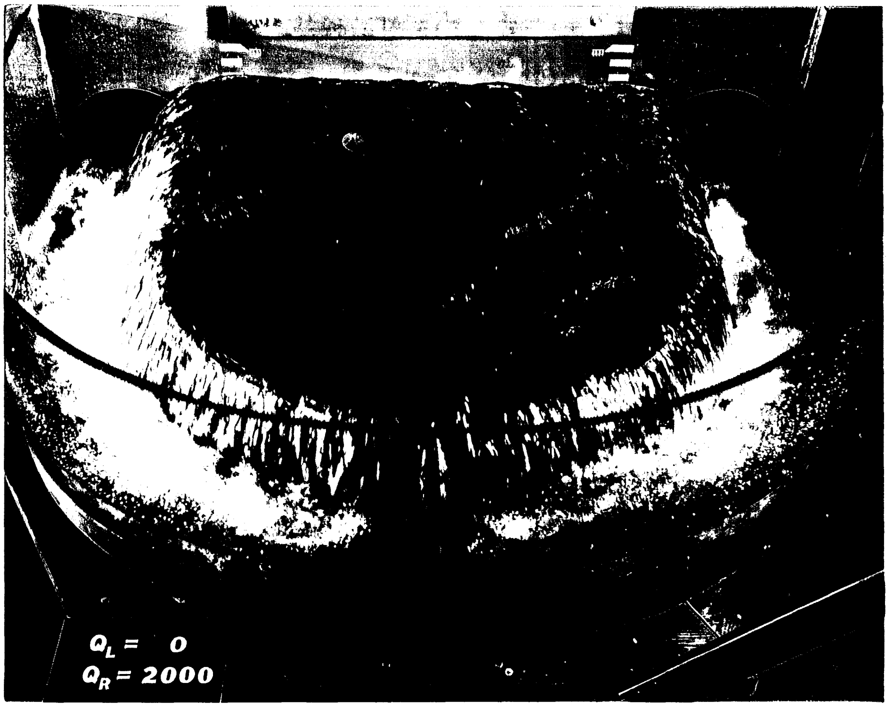


Photo 17. Flow over relief weir, Inflow Q_R = 2000, Q_L = 0; Outflow Q_R = 1000; Q_L = 1000 cfs.

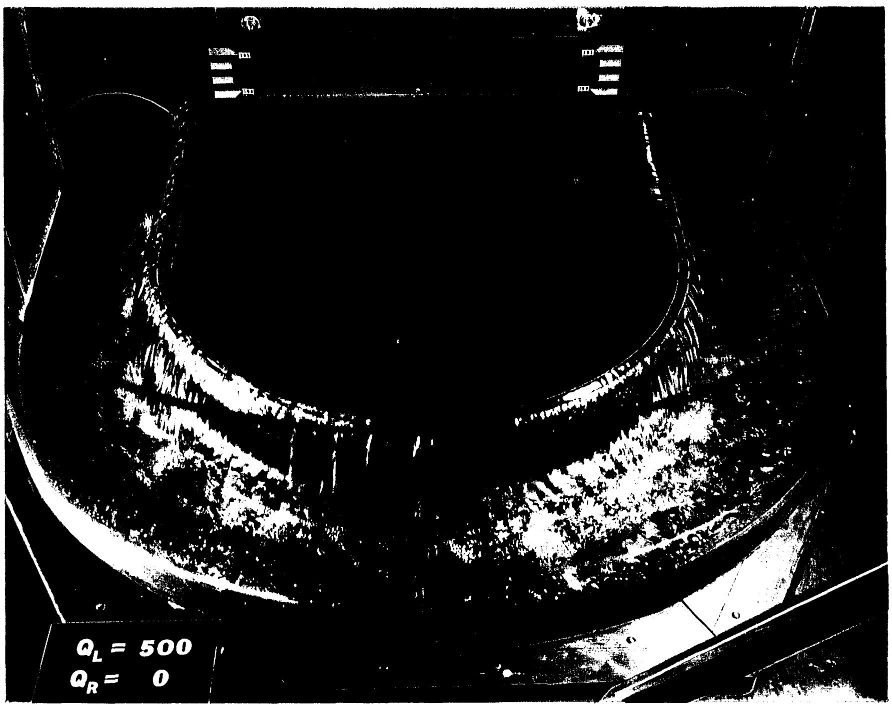


Photo 18. Flow over relief weir, Inflow Q_R = 0, Q_L = 500; Outflow Q_R = 250, Q_L = 250 cfs.

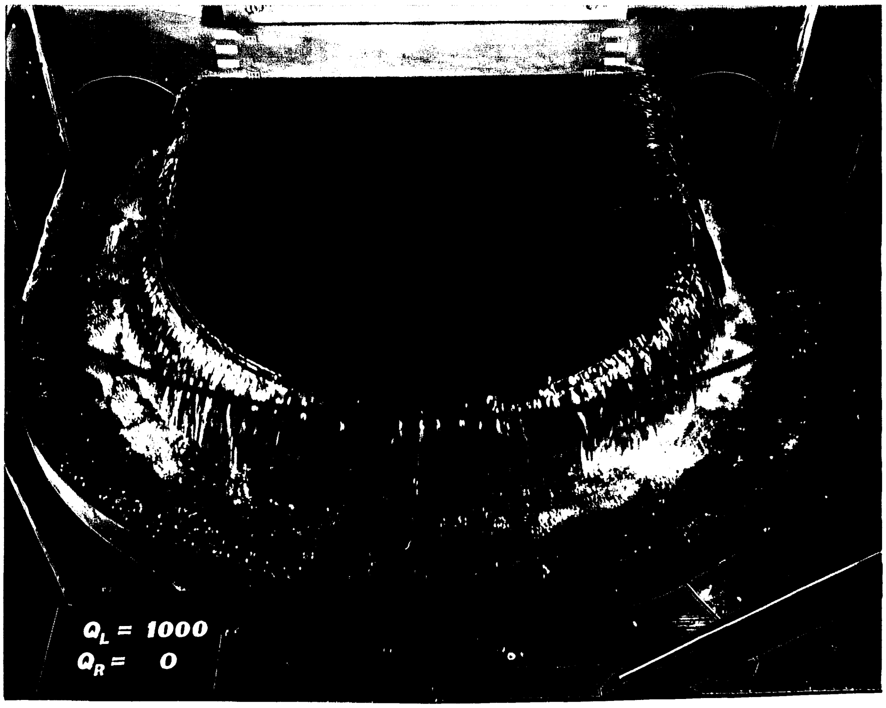


Photo 19. Flow over relief weir, Inflow $Q_R = 0$, $Q_L = 1000$; Outflow $Q_R = 500$, $Q_L = 500$ cfs.

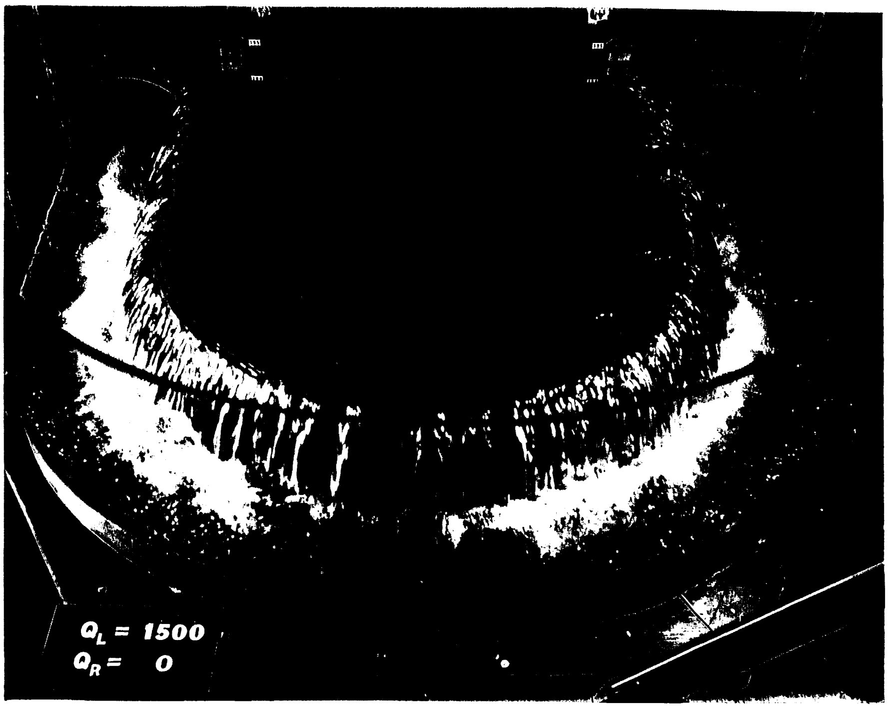


Photo 20. Flow over relief weir, Inflow $Q_R = 0$, $Q_L = 1450$; Outflow $Q_R = 750$, $Q_L = 700$ cfs.

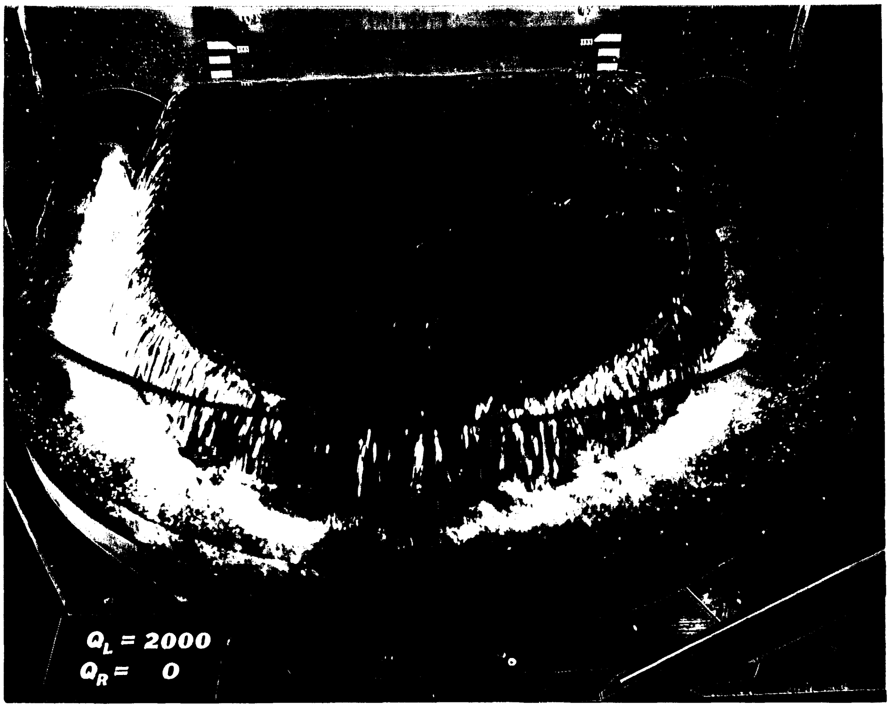


Photo 21. Flow over relief weir, Inflow $Q_R = 0$, $Q_L = 1950$; Outflow $Q_R = 950$, $Q_L = 1000$ cfs.

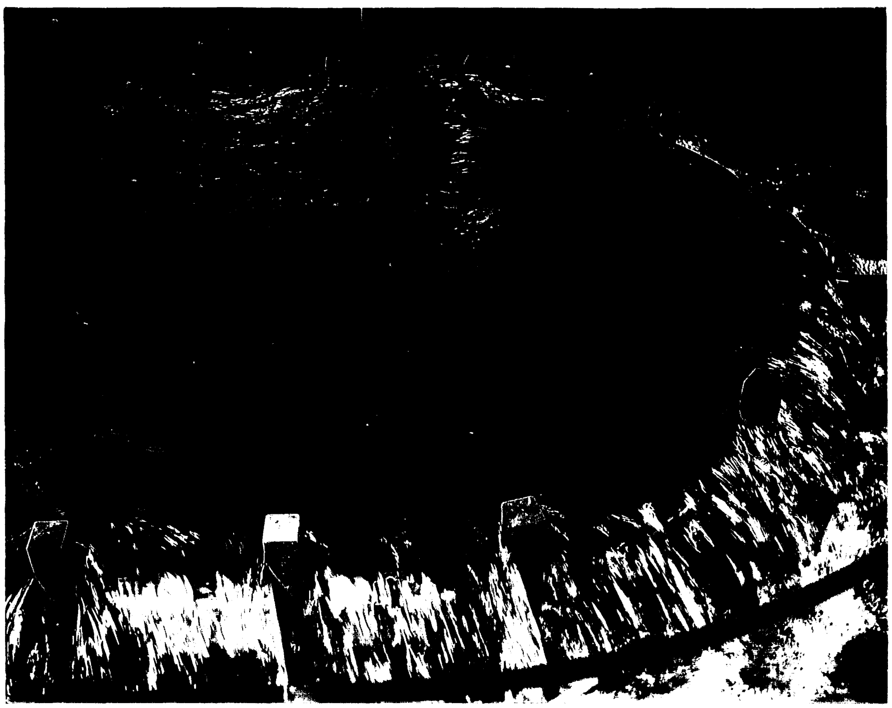


Photo 22. Weir crest showing flow splitters, $Q_R = Q_L = 1500$ cfs.

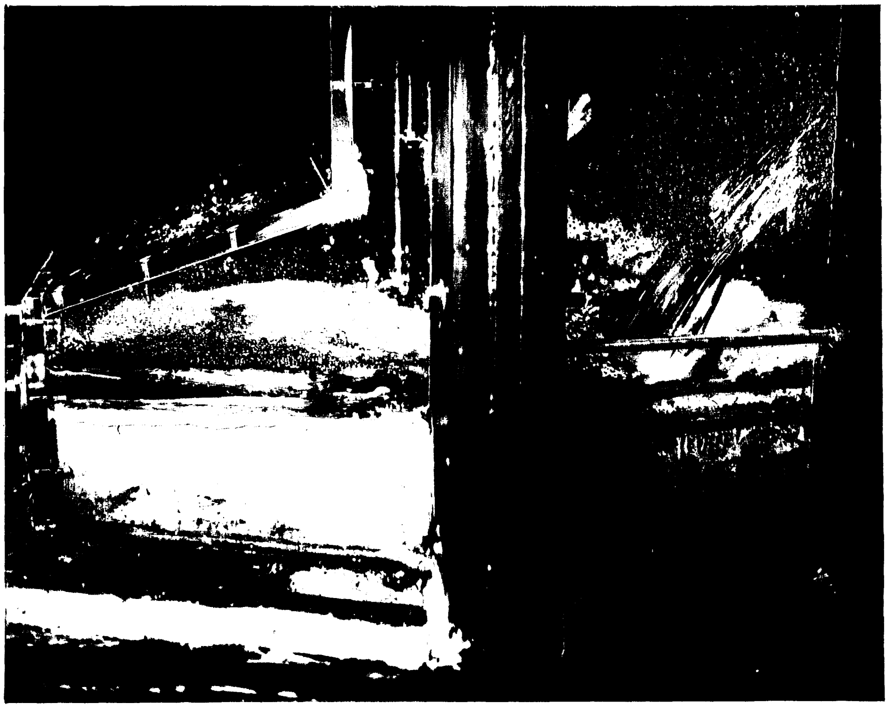


Photo 23. Flow in head-breaker and transition chamber. Middle gates full open. $Q_R = Q_L = 187.5$ cfs, W.S. elevation = 365.3.

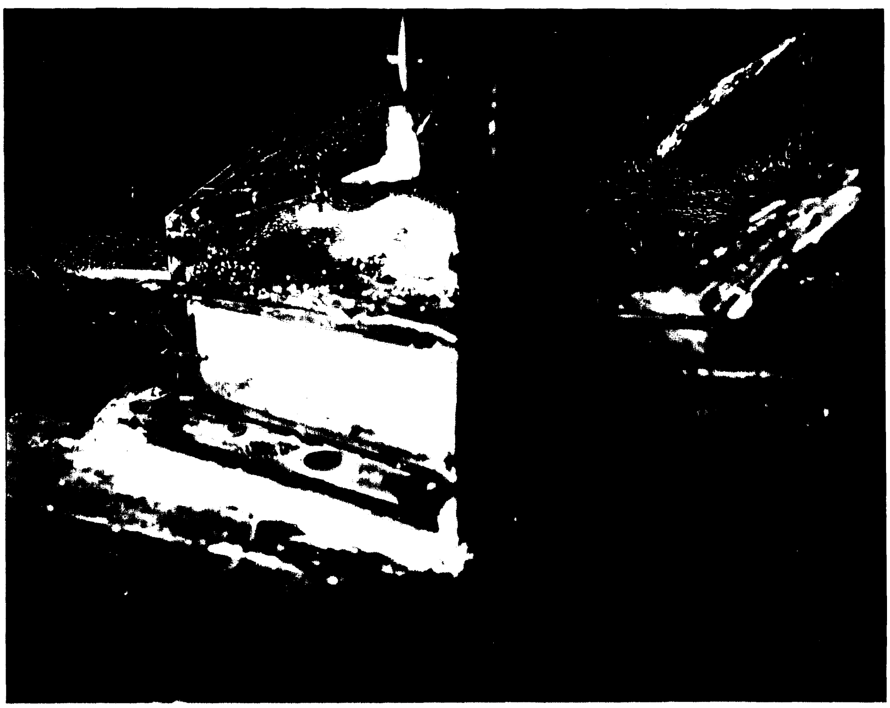


Photo 24. Flow in head-breaker and transition chamber. Middle gates 2.54 ft open. $Q_R = Q_L = 187.5$, W.S. elevation = 370.3.

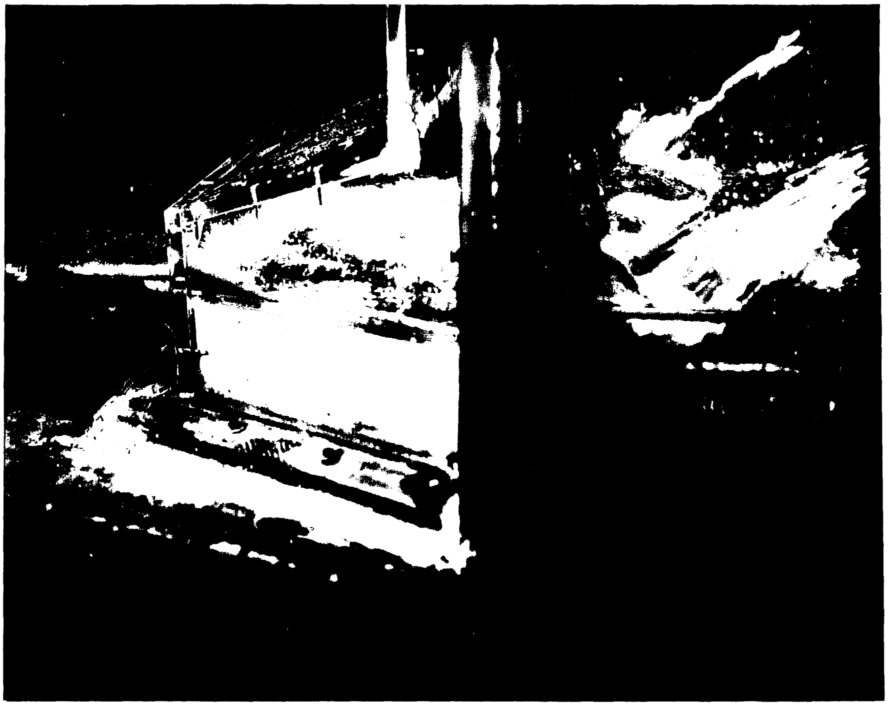


Photo 25. Flow in head-breaker and transition chamber. Middle gates 2.04 ft open. $Q_R = Q_L$ = 187.5, W.S. elevation = 375.3.

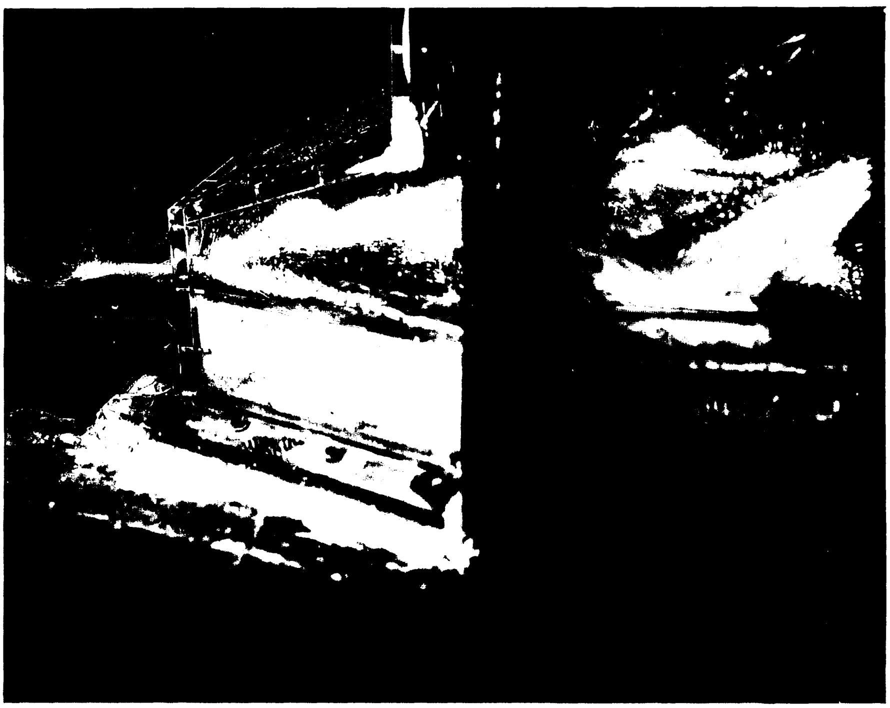


Photo 26. Flow in head-breaker and transition chamber. Middle gates 1.83 ft open. $Q_R = Q_L$ = 187.5 cfs, W.S. elevation = 380.3.

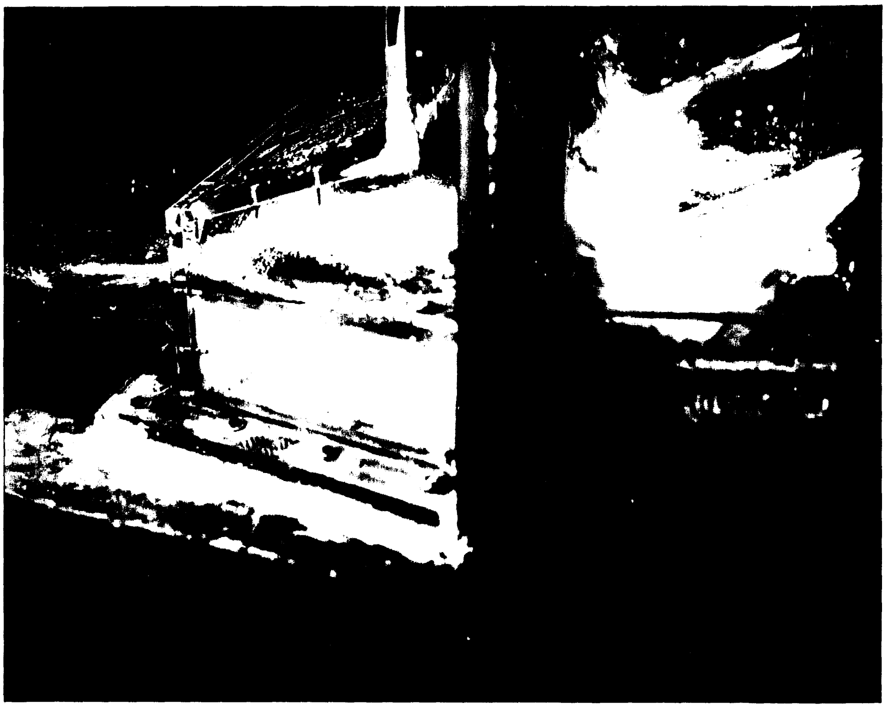


Photo 27. Flow in head-breaker and transition chamber. Middle gates 1.5 ft open. $Q_R = Q_L = 187.5$ cfs, W.S. elevation = 385.33.

Photo 28. Flow in head-breaker and transition chamber. Top gates full open. $Q_R = Q_L = 187.5$, W.S. elevation = 378.0.

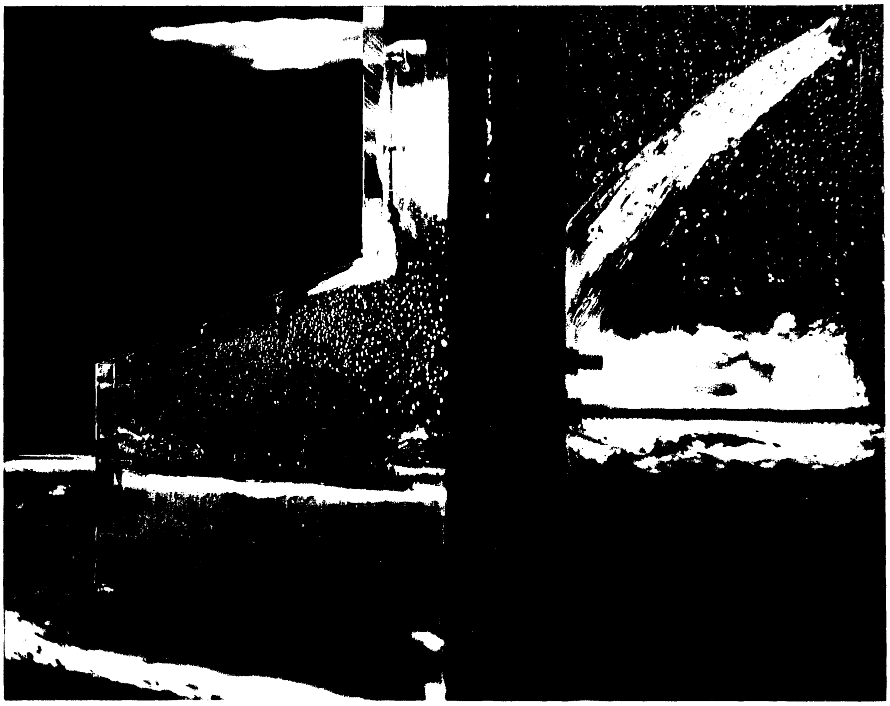


Photo 29. Flow in head-breaker and transition chamber. Top gates 2.24 ft open. $Q_R = Q_L = 187.5$, W.S. elevation = 383.0.

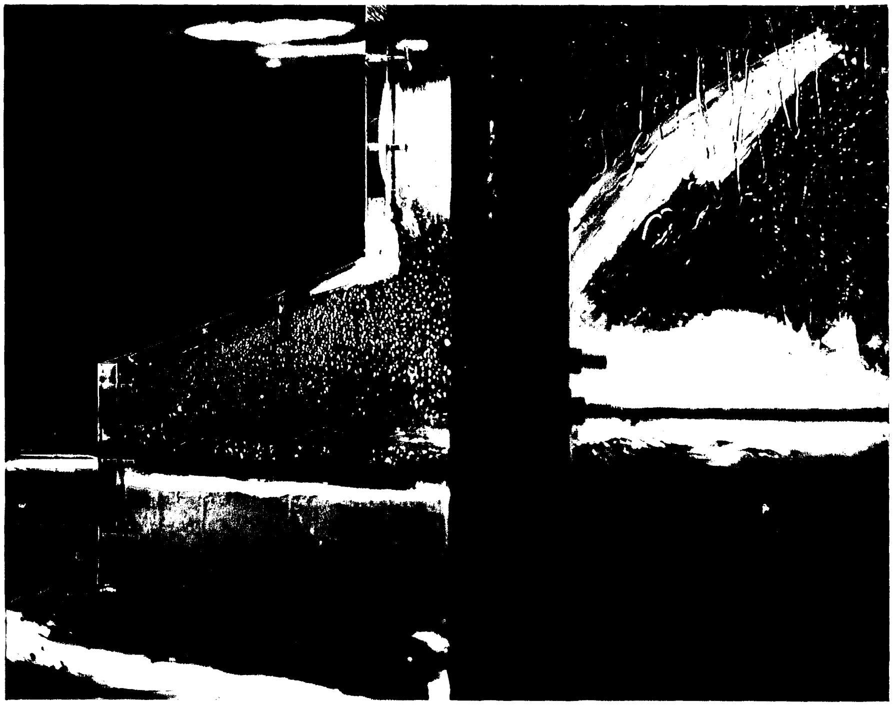


Photo 30. Flow in head-breaker and transition chamber. Top gates 1.94 ft open. $Q_R = Q_L = 187.5$, W.S. elevation = 388.0.