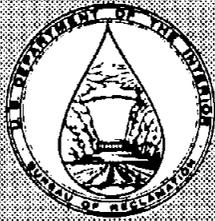


R-89-12



# UNION AVENUE DAM BOATCHUTE STUDY



September 1989

**U.S. DEPARTMENT OF THE INTERIOR**  
Bureau of Reclamation  
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# UNION AVENUE DAM BOATCHUTE STUDY

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September 1989



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The team conducting the study from the Hydraulics Branch, Bureau of Reclamation, was comprised of Clifford A. Pugh, Head, Hydraulic Equipment Section; Cassie Klumpp, Hydraulic Engineer; and Jerry R. Fitzwater, Civil Engineering Technician.

The laboratory shops did an excellent job of constructing the model. Most of the photographs in this report were taken by Mr. Wayne Lambert, staff photographer.

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## CONTENTS

	Page
Introduction .....	1
Conclusions and recommendations .....	1
The model .....	2
Description .....	2
Model measurements and instrumentation .....	2
Hydraulic similitude .....	3
Sediment scaling .....	4
Model results and analysis .....	5
Model calibration .....	5
Calibration of dam, radial gate, and intake .....	6
Study of boating conditions .....	6
Final boatchute configuration .....	8
Flow patterns .....	9
Sediment studies .....	10
Sluicing flows .....	10
Approach flow to Englewood intake .....	10
Sediment tests .....	11
Floodflows .....	12
Flow over the main crest .....	12
Lowering the sluice wall .....	13
Riprap and channel stability .....	13
Bibliography .....	13

## TABLES

Table		
1	Average monthly South Platte riverflows at Littleton, Colorado, and average Englewood diversions .....	15
2	Velocity comparison in front of primary Englewood water intake with high and low walls .....	15
3	Water surface elevations from Union Avenue bridge to the pool downstream of boatchute 2 .....	16

## CONTENTS - Continued

### FIGURES

Figure		Page
1	Section through existing Union Avenue dam showing typical reverse roller hydraulic jump .....	17
2	Kayaker at Brown's Ditch dam boatchute .....	17
3	Recommended boatchute configuration .....	18
4	Section along centerline of Union Avenue dam boatchute (boatchute 1) .....	19
5	Sections through Union Avenue dam boatchute .....	20
6	Model study area .....	21
7	Settling velocity of sand and silt in water .....	22
8	Model and prototype grain size distribution for bedload .....	23
9A	Union Avenue dam discharge rating curve (with boatchute) .....	24
9B	Union Avenue dam discharge rating curve (with boatchute) .....	24
10	Union Avenue dam radial gate discharge rating curves with boatchute .....	25
11	Englewood intake discharge curves .....	25
12	Boatchute 1 constructed in model - original design .....	26
13	Boatchute 2 - original design .....	26
14	Open bar-type barricade constructed along Englewood intake .....	27
15	Union Avenue dam boatchute 1 narrowed with rocks .....	27
16	Boatchute 2 narrowed with rocks .....	28
17	Union Avenue dam modification of boatchute with single ramp in center of chute .....	28
18	Section through existing spillway showing modifications with wedge block and rockfill on downstream face .....	29
19	Design with two ramps in center of boatchute 1 .....	29
20	Boatchute 1 with ramp extending across boatchute, no center trough along ramps .....	30
21	Kayak proceeding through boatchute .....	30
22	Boatchute 1 with trough through first ramp .....	31
23	Another view of first boatchute .....	31
24	V-wave pattern in first boatchute .....	32
25	Boat proceeding through first boatchute .....	32
26	Boatchute 2 .....	33
27	Water surface vs. discharge in pool between boatchutes 1 and 2 .....	33
28	Water surface elevation vs. discharge between boatchutes 2 and 3 .....	34
29	Overall view of model with final design at 1,500 ft <sup>3</sup> /s. ....	34
30	Boatchute 1 at 500 ft <sup>3</sup> /s .....	35
31	Boatchute 1 at 1,500 ft <sup>3</sup> /s .....	35
32	Boatchute 2 at 500 ft <sup>3</sup> /s .....	36
33	Boat going through boatchute 2 at 1,500 ft <sup>3</sup> /s .....	36

## CONTENTS - Continued

Figure		Page
34	Surface velocities in approach channel to Union Avenue dam, grid spacing = 18 feet, $\Delta t = 8.5$ seconds, $Q = 1,000$ ft <sup>3</sup> /s .....	37
35	Surface velocities in approach channel to Union Avenue dam, grid spacing = 18 feet, $\Delta t = 8.5$ seconds, $Q = 1,500$ ft <sup>3</sup> /s .....	38
36	Surface velocities of boatchute 1, grid spacing = 18 feet, $\Delta t = 3.4$ seconds, $Q = 500$ ft <sup>3</sup> /s .....	39
37	Surface velocities in pool downstream of boatchute 1, grid spacing = 8 feet, $\Delta t = 3.4$ seconds, $Q = 1,000$ ft <sup>3</sup> /s .....	40
38	Surface velocities downstream of boatchute 1, grid spacing = 18 feet, $\Delta t = 3.4$ seconds, $Q = 1,500$ ft <sup>3</sup> /s .....	41
39	Streak lines in first pool between boatchutes 1 and 2, $Q = 1,500$ ft <sup>3</sup> /s .....	42
40	Boating conditions with open-bar barrier in front of primary Englewood intake .....	43
41	Approach flow velocities upstream of Englewood intake ( $Q = 240$ ft <sup>3</sup> /s) .....	44
42	Approach flow velocities upstream of Englewood intake ( $Q = 811$ ft <sup>3</sup> /s) .....	45
43	Approach flow velocities upstream of Englewood intake ( $Q = 1,450$ ft <sup>3</sup> /s) .....	46
44	Area between two boatchutes prior to sediment test .....	47
45	Overhead view of area between boatchutes prior to sediment test .....	47
46	Contours of pool area between boatchutes 1 and 2 prior to sediment test .....	48
47	Contours of pool area between first and second boatchutes after to sediment test .....	48
48	Area between boatchutes after sediment testing .....	49
49	Deposition after sediment test .....	49
50	Dunes formed upstream of the boatchute 1 after sediment test .....	50
51	Model with flow at 16,400 ft <sup>3</sup> /s (100-year flood) .....	50
52	Water surface profiles through boatchute 1 .....	51
53	Section through Union Avenue dam showing reduction in rockfill from original boatchute design .....	51

## INTRODUCTION

In 1985, the COE (U.S. Army Corps of Engineers) constructed a dam 300 feet downstream of Union Avenue in Englewood on the South Platte River. The dam serves as a diversion structure for a water supply intake for the city of Englewood and also controls the grade of the South Platte River for flood control. The reinforced concrete dam is 18.5 feet high and has a 3 to 1 slope on the face of the spillway. The total head drop is 15 feet to the downstream channel. A section through the existing dam is shown on figure 1. The energy is dissipated in a hydraulic jump at the base of the spillway. This type of energy dissipator has been used successfully at hundreds of structures throughout the world. However, the reverse roller in the jump can trap a boat or person and is hazardous when a boat goes over the dam. Recreational boating has become more popular on the South Platte in recent years (fig. 2); therefore, boater safety has become a primary concern.

The CWCB (Colorado Water Conservation Board) is investigating structural alternatives to improve boater safety at the Union Avenue dam. One alternative is to construct a series of boatchutes, each with a drop of 2 to 4 feet, to handle the entire 15-foot drop. The CWCB selected WWE (Wright Water Engineers, Inc.) to plan the boatchutes at the site. The CWCB also requested assistance on the project from Reclamation (the Bureau of Reclamation). Reclamation agreed to construct a physical model of the boatchute in the Hydraulic Laboratory. Funding for the model study was provided primarily by the Great Plains Region's Assistance to States Program; the CWCB provided partial funding for the work. The testing was divided into three phases:

- Phase A. - Model tests to achieve desired flow patterns for boating at discharges between 100 and 1,500 ft<sup>3</sup>/s.
- Phase B. - Sediment tests to observe deposition and scour patterns as they affect the Englewood water intake structure upstream of Union Avenue dam and deposition and scour patterns in the boatchute 1 pool.
- Phase C. - Floodflow tests up to the 100-year flood of 16,400 ft<sup>3</sup>/s.

## CONCLUSIONS AND RECOMMENDATIONS

1. A double-ramp boatchute configuration was developed as a result of the model study. This boatchute configuration eliminates dangerous reverse roller hydraulic jumps (figs. 3-5) and provides boatable waves downstream throughout the entire flow range. Boating safety will be greatly improved by the addition of the proposed features.
2. Guidelines were developed for designing the boatchutes. These guidelines can also be applied to other sites with head drops between 2 and 3.5 feet.
3. With the present Englewood water intake design, boaters would be drawn into the intake area during sluicing.
4. A solid wall [the same height as the intake wall (elevation 5296.5)] is recommended in front of the primary Englewood intakes to protect boaters and to enhance the present sluicing capacity (fig. 3).

5. A recommended location for an open bar barrier to prevent boaters from entering the water intake area upstream of the Englewood intake was determined.
6. As a result of this study, the volume of rockfill for the Union Avenue dam modification was reduced by 1,500 yd<sup>3</sup> over the initial design. Downstream rockfill embankments can also be reduced in volume compared to the initial design.
7. The need to raise the left sluiceway wall to the right of boatchute 1 was eliminated.
8. Sediment tests indicated that the Englewood intake with the recommended high wall will sluice sediment better than the present design.
9. Sediment deposition patterns in the first pool between boatchutes 1 and 2 were determined. The deposits will not affect sluicing operations.
10. Flow velocities and directions in the pool are acceptable for boating before or after the deposition occurs.
11. A long wedge block (fig. 18) was added along the downstream face of Union Avenue dam to eliminate a reverse roller that could trap boaters when the main dam crest is overtopped. The discharge coefficient of the crest was unaffected by the wedge.
12. During the 100-year flood, only minor movement of 3-foot riprap occurred on the downstream face of the dam. An area to the right of boatchute 1 should be stabilized to prevent erosion.
13. Addition of the boatchutes will not adversely affect the Englewood intake structure.
14. Discharges up to the COE 100-year flood of 16,400 ft<sup>3</sup>/s were observed in the model. Model data indicate that the water level is contained within the streambanks up to the 100-year flow.

## **THE MODEL**

### **Description**

A 1 to 18 model scale was chosen primarily due to the need to simulate sediment transport and to study boating flows as low as 100 ft<sup>3</sup>/s. The model features included the Union Avenue dam, city of Englewood intake structure, radial-gate-controlled sluiceway, downstream pool, and the second rockfill dam and boatchute (fig. 6). The large bend in the river upstream from the bridge was included to duplicate flow conditions at the Englewood intake structure.

### **Model Measurements and Instrumentation**

The model was calibrated to obtain stage-discharge curves over the dam, through the Englewood sluice, and through the Englewood intake.

Point gauges were used to measure water surface elevations in the pool upstream of the crest, through the boatchute, in the pool between the boatchutes 1 and 2 and the pool after boatchute 2.

Flows less than 5,000 ft<sup>3</sup>/s (3.6 ft<sup>3</sup>/s model) were supplied by a portable pump and measured with a Venturi orifice meter and a differential mercury manometer. For flows exceeding 5,000 ft<sup>3</sup>/s, water was delivered to the model by the laboratory control supply and measurement system.

Velocity measurements were taken using Marsh McBirney electromagnetic flowmeters and Ott current meters. Surface velocities and directions were obtained by taking time lapse pictures of model boats traversing the area.

### Hydraulic Similitude

Hydraulic similitude must exist between model and prototype to obtain accurate flow measurements in the model. The inertial force is the vector sum of all forces. When gravitational forces predominate, which is the case with most open channel hydraulic structures, a basis for similitude can be established by equating the ratio of gravitational forces to inertial forces and neglecting the other forces. Flow in the model was simulated by using the dimensionless Froude number which relates inertial force to gravity force.

$$F_r = \frac{V}{\sqrt{Lg}} \quad (1)$$

Using the Froude number, model and prototype parameters can be determined from the following similitude equations:

$$L_r = \frac{L_p}{L_m} = 18 \quad (2)$$

$$A_r = L_r^2 = 324 \quad (3)$$

$$V_r = L_r^3 = 5,832 \quad (3)$$

$$T_r = L_r^{1/2} = 4.24 \quad (5)$$

$$V_r = L_r^{1/2} = 4.24 \quad (5)$$

$$Q_r = (L_r)^{5/2} = 1,375 \quad (7)$$

Where:

- $L_r$  = length ratio
- $A_r$  = area ratio
- $V_r$  = volume ratio
- $T_r$  = time ratio
- $V_r$  = velocity ratio
- $Q_r$  = discharge ratio

Subscripts m and p refer to the model and prototype, and r is the ratio between model and prototype.

### Sediment Scaling

Sediment models that involve erosion of noncohesive bed material must simulate shear stress ( $\tau_o$ ) because the shear stress creates the drag force required to overcome the forces holding a particle in place. Shear stress on a particle will fluctuate because of turbulence. Drag force and turbulence are a function of Reynolds number. A model operated according to Froude scaling does not necessarily simulate tractive forces and sediment erosion accurately. Sediment erosion can be simulated properly by making the model and prototype dimensionless unit sediment discharge rates equal ( $q_{s,m} = q_{s,p}$ ).

The following equations define dimensionless shear stress ( $\tau'$ ), Grain Reynolds number ( $R'$ ) and dimensionless unit sediment discharge ( $q_s'$ ). These equations are used to relate model and prototype parameters to determine sediment erosion characteristics.

$$\tau' = \frac{u^2}{gd} \frac{\gamma}{(\gamma_s - \gamma)} \quad [\text{dimensionless shear stress}] \quad (8)$$

$$R' = \frac{u'd}{\nu} \quad [\text{Grain Reynolds number}] \quad (9)$$

$$q_s' = \frac{q_s}{u'd} \quad [\text{dimensionless unit sediment discharge}] \quad (10)$$

$$u' = v \sqrt{\frac{f}{8}} \quad [\text{shear velocity}] \quad (11)$$

Where:

- f = the Darcy friction factor
- d = the sediment particle size
- u' = the shear velocity
- $\nu$  = the kinematic viscosity
- $\gamma$  = specific weight of water
- $\gamma_s$  = specific weight of sediment

Shields developed a diagram relating dimensionless shear stress to Grain Reynolds number (Vanoni, 1975). Shields used this diagram to define critical shear stress. Vanoni (1975) used Taylor's data to show that dimensionless unit sediment discharge at low transport levels falls very close to Shields curve for incipient motion. In order to properly model sediment transport, the dimensionless unit sediment discharge rate ( $q_s'$ ) must be the same in model and prototype. Details of scaling sediment transport are outlined in the report "Hydraulic Model Studies of Fuse Plug Embankments" (Pugh, 1985). Dimensionless shear stress is a form of the Froude number and the density ratio of sediment to water. If a model is scaled geometrically according to Froude scaling ( $\tau_m' = \tau_p'$ ), the model unit sediment discharge rate ( $q_s'$ ) will be too great for Grain Reynolds numbers ranging between 5 and 100. Therefore, the model sediment size fractions should be adjusted to properly simulate sediment transport in this range.

A diagram of settling velocity ( $w$ ) of sand and silt particles (fig. 7) illustrates that small particles (less than 1 mm in diameter) settle at slower velocities as the particles become smaller. In order to adjust the sediment discharge rate in the model, particles less than 1 millimeter in diameter are increased in size until the settling velocity is corrected to the proper velocity consistent with Froude scaling. Particles larger than 1 millimeter settle as a function of the diameter ( $d$ ) to the 1/2 power, consistent with Froude scaling. When the model grain sizes are adjusted for settling velocity, the value of  $\tau$  decreases while the value of  $R$  increases bringing the model value of  $q_s'$  closer to the same value as the prototype. The grain size distribution of bedload in the South Platte at Oxford Avenue was simulated in the model. Figure 8 shows the prototype and model grain size distributions. The model grain sizes were adjusted as described above to compensate for Reynolds number effects.

## MODEL RESULTS AND ANALYSIS

The main areas of the model study were (1) model calibration, (2) boating conditions, (3) sediment studies, and (4) floodflow conditions. These and other areas are discussed in the following sections.

### Model Calibration

The model was calibrated to accurately measure discharge. Flow entering the model was measured with a Venturi orifice meter. Orifice plates ranging in size from 1-3/8 to 4-3/8 inches were used. A mercury manometer indicated the differential across the plates. The accuracy of the orifice meter and manometer was also checked with a strap-on sonic flowmeter. The comparison of the orifice with the flowmeter showed less than a 2-percent difference in flow measurement indicating no problem with measurement of flow entering the model.

Volumetric calibrations were also performed to compare to the orifice meter readings. A known volume in the head box of the model was filled and timed to obtain a discharge. This calibration was within 1 percent of the orifice meter. These calibrations were necessary since the flows being modeled were very small – as low as 50 ft<sup>3</sup>/s (0.036 ft<sup>3</sup>/s model). Care was taken during model construction to prevent leakage through the Union Avenue dam and at the downstream rock dam. Checks after the model was built confirmed that very little leakage occurred.

**Calibration of dam, radial gate, and intake.** - The Union Avenue dam was calibrated for flow vs. water surface elevation by taking point gauge readings of the water surface approximately 30 feet upstream of the boatchute. A discharge rating curve of the Union Avenue dam and boatchute is presented on figures 9A and 9B. Two regression equations are presented on figure 9A, depending on the elevation of the water surface. The breaks in the curve correspond to the configuration of the boatchute in relation to the water surface elevation. Figure 9B shows an enlargement of the lower portion of the discharge rating curve.

The Englewood sluice radial gate was also calibrated for flow vs. water surface. Water surface readings were made at the boatchute centerline about 30 feet upstream of the Union Avenue dam for different gate openings between 1/2 foot and 3 feet. The flow over the dam (figs. 9A and 9B) was subtracted from the total inflow to obtain the radial gate flow. Figure 10 shows discharge rating curves for different gate openings.

The Englewood water intake model flow was also calibrated. Flow through the Englewood intake was measured volumetrically. The number of turns on a valve controlling the model flow through the intake was related to the water surface elevation (fig. 11). The model was normally operated with the valve open two turns representing 30 ft<sup>3</sup>/s. This operation was necessary to ensure the correct distribution of flow between the Englewood intake, the sluice, and the flow over the boatchute and dam. Table 1 presents a typical distribution of flows that Englewood takes throughout the year. During the boating season (May-September), Englewood's withdrawal is typically 25 to 30 ft<sup>3</sup>/s.

### **Study of Boating Conditions**

WWE provided Reclamation with drawings to construct the original boatchute configuration in the physical model (figs. 12 and 13). Boatchute 1 was 32 feet wide and the invert elevation was 5288 feet. Boatchute 2 downstream was also 32 feet wide and the invert elevation was 5284.5 feet. The effect of the third boatchute on the upstream pool elevations was simulated by incorporating a weir in the downstream end of the model with the proper shape and elevations to simulate boatchute 3. Boatchute 1 curved toward the left to attempt to make the boats approach boatchute 2. Tests were conducted for boating flows ranging between 100 and 3,000 ft<sup>3</sup>/s. The results of the tests showed that the original boatchute configuration was not satisfactory for boating flows. Rafts and kayaks impinged on the right side of the boatchute. Boatchute 2 handled boats better, but some design changes were also necessary to further improve the flow.

In January 1989, a revised boatchute design was constructed and tested in the physical model. The Union Avenue dam boatchute was 32 feet wide and boatchute 2 was 64 feet wide. Both chutes employed a center trough to concentrate flow for smaller boating flows. An open bar barricade was placed along the Englewood intake (fig. 14) to prevent boaters from entering the sluiceway. The left side of the boatchute was lowered (causing a superelevated chute) to try to force flow to the

left. Flows ranging between 100 ft<sup>3</sup>/s and 1,500 ft<sup>3</sup>/s were tested in the model. Boats moved toward the sluice wall while passing through the Union Avenue dam boatchute.

Reclamation conducted two demonstrations of the second design configuration. The first demonstration on January 18, 1989, showed the model operating at a range of flows between 100 and 16,400 ft<sup>3</sup>/s. Attendance at the first demonstration included personnel from the cities of Englewood, Bowmar, and Sheridan; CWCB; UDFCD (Urban Drainage and Flood Control District); COE; and WWE.

During the first demonstration, both chutes were narrowed by placing large boulders (5-foot prototype) along each side of the chute to help boating conditions at smaller flows. Photographs of this flow condition are shown on figures 15 and 16. Narrowing the boatchutes improved lower flow conditions, but an undesirable wave still existed downstream of the Union Avenue dam boatchute. The boats had a tendency to turn toward the sluice wall while going through boatchute 1. A high center wave also existed downstream of boatchute 2.

The wave height was adjusted by placing a ramp in the center of the boatchute. The purpose of the ramp was to prevent boaters from taking on bow water by reducing the center wave. Initially, a single ramp 15 feet long and 7.5 feet wide was tested in the model (fig. 17). The single ramp reduced the height of the wave and caused a more desirable wave pattern for boating. However, the wave was still too high for safe boating conditions at higher flows at Union Avenue dam boatchute. A long wedge-shaped block was placed on the downstream face of the Union Avenue dam to the left of the boatchute. The wedge blocks will alleviate the dangerous roller that developed over the dam when flows exceeded 500 ft<sup>3</sup>/s (fig. 18). The wedge block placed on the face of the dam greatly improved flow conditions over the dam and was recommended for final design. The block was placed far enough down the dam face to avoid altering the discharge coefficient of the crest. If the discharge coefficient was altered, the river would not remain within its banks at the 100-year flood due to the reduction in efficiency.

During the second demonstration (January 31, 1989), the model was run for flows ranging from 100 to 8,000 ft<sup>3</sup>/s. A video tape of the 100-year flood of 16,400 ft<sup>3</sup>/s was shown to the participants.

Several design changes were attempted on boatchute 1. The height of the rocks on the left side of Union Avenue boatchute was raised to prevent backflow into the boatchute from the left. This change improved the boating flow conditions. The wave moved up on the ramp, and boats turned left toward boatchute 2 instead of heading toward the sluice wall. However, slight changes in the placement of the rocks would affect the flow, and the proximity of the riprap to the boatchute on the left side was a safety concern.

Reduction in the height of the roller on boatchute 1 was accomplished by using a combination of two ramps in the model. One ramp was set at elevation 5287.71, and the lower ramp was set at 5286.24 feet. Initially, the ramps only extended across the center of the boatchute (fig. 19), and rocks still extended along either side of the boatchute. This boatchute configuration reduced the height of the wave at the bottom of the boatchute, but boats still turned toward the sluice wall unpredictably.

The next design change attempted in the model included two ramps completely across the boatchute with a center trough up to the first ramp (figs. 20 and 21). Flow was uniform through the

boatchute, but boats did not pass through at low flows without hitting the first ramp. Boats tended to turn sideways as they went over the second ramp. This configuration significantly reduced the wave heights.

The next boatchute design configuration constructed in the physical model for boatchute 1 included two ramps with a center trough extending through the first ramp but not through the second ramp (figs. 22 and 23). The elevation of the upper ramp was reduced to 5287.25 feet and the second ramp was increased to 5286.5 feet. Wave patterns were acceptable for boating at 100 ft<sup>3</sup>/s and 800 ft<sup>3</sup>/s with a V-wave pattern (figs. 24 and 25); however, the wave in the center was larger than the previous configuration. The boats remained straight as they passed through the chute.

A third demonstration was held on March 22, 1989, with personnel attending from Reclamation, WWE, COE, CWCB, UDFCD, GS (Geological Survey), and the city of Englewood. The model was demonstrated for flows between 100 and 8,000 ft<sup>3</sup>/s. UDFCD proposed a change in the alignment of the second dam and boatchute 2 and reorientation of boatchute 2. Boatchute 2 was directed away from the left bank to prevent development of a scour hole and possible bank erosion. The right side of the second dam was brought up to elevation 5287 to form a wedge shape and prevent erosion along the right bank.

Boatchute 2 was reconstructed in the model with the invert elevation at 5284.5 feet (fig. 26). Boatchute 2 contained a center trough through the first ramp (the same as boatchute 1). Wave patterns were excellent with a V-wave forming in the middle of the second ramp.

Tests were run on boatchute 2 to determine the optimum elevation of the ramps with respect to the tailwater elevation. The elevation of the weir blade on the end of the model was adjusted to simulate elevation changes at boatchute 3 (not modeled).

The final design modification occurred in May 1989 on boatchute 1. Using information obtained from the optimization tests on boatchute 2, the elevation of the first ramp was reduced to 5286.75 feet, and the elevation of the second ramp was reduced to 5286.0 feet. The length of the 10 to 1 slope was increased to accommodate the reduction in elevation of the two ramps (figs. 3-5). This modification was made to accommodate the larger drop of 3-1/2 feet at boatchute 1. Boatchute 2 drop was 2-1/2 feet. The wave patterns and boating conditions were excellent throughout the boating flows.

**Final boatchute configuration.** - After the final configuration of the boatchute was established, including the low flow notch in the center and the combination of two ramps to disperse the wave, a series of tests was conducted to determine the optimum elevation of the ramps with respect to tailwater elevation. Figure 27 shows the optimum relationship between the ramp elevations on boatchute 1, boatchute 2, and the pool between the two boatchutes. The capacity of the low flow notch in boatchute 2 (elevation 5284.5 to 5286.0) is approximately 30 ft<sup>3</sup>/s. For flows exceeding 30 ft<sup>3</sup>/s (including sluice flows), the second ramp in boatchute 1 will be completely submerged. The low flow notch extends through the first ramp; therefore, small boats will be able to pass through the notch at low flows. As the flow increases and the pool rises, the combination of the two ramps spreads the wave while maintaining a V pattern in the center. Without the low flow notch through the first ramp, the wave was uniformly dispersed in the downstream pool and the wave height was reduced. However, boats tended to turn sideways as they went over the second ramp. The low flow notch maintains a V-wave in the center which keeps the boats straight. Pilot rocks placed upstream

of and outside the 32-foot-wide boatchute will not adversely affect flow patterns. However, a pilot rock should not be used on the right side of boatchute 1 since space is not available in this area.

The final design of the boatchutes was optimized for a riverflow of 500 ft<sup>3</sup>/s; however, the wave characteristics are acceptable throughout the entire range of boating flows from 50 ft<sup>3</sup>/s to 3,000 ft<sup>3</sup>/s.

The relative elevations between the ramps in boatchute 2 and the weir elevations in boatchute 3 are the same as those between boatchutes 1 and 2 (fig. 28) even though the head drop over the second chute was less than over the first. The model tests indicate that the elevations of the ramps in the upstream chute can be off by  $\pm 4$  inches and the wave characteristics will still be adequate. The configuration reported provides optimum performance.

These guidelines should be followed when using the standard boatchute design developed in this model study:

1. The second ramp in the boatchute should be placed at an elevation 0.4 foot above the design elevation (defined in fig. 27) of the downstream control.
2. Both ramps should be 10 feet long and 0.75 foot high; the top of the ramps should be horizontal.
3. The slope of a line connecting the crest of the dam with the lip of the second ramp should be 1 to 10 (fig. 4).
4. A low flow notch should extend from the crest through the first ramp.

The general boatchute configuration is acceptable for head drops from 2 to 3.5 feet. The length of the boatchute will increase as the head drop increases.

Both boatchutes were tested in the laboratory for river discharges ranging between 100 and 3,000 ft<sup>3</sup>/s. Photographs of boatchute operation with final design configuration are shown on figures 29 through 33.

**Flow patterns.** - Time lapse photographs of 8-foot boats approaching Union Avenue dam and traveling between boatchutes 1 and 2 were used to define flow patterns. Lights on the boats produced streak lines. The grid spacing was 18 feet prototype. A strobe unit flashing at constant time intervals ( $\Delta t$ ) produced images of the boats. The velocity of a boat during each interval can be determined by dividing the distance the boat traveled by  $\Delta t$  for the photograph.

Surface velocities and flow lines are shown on figures 34 and 35 in the approach channel to Union Avenue dam at flows of 1,000 and 1,500 ft<sup>3</sup>/s. Surface velocities and flow lines in the pool between boatchutes 1 and 2 are shown on figures 36 through 38 for discharges ranging between 500 and 1,500 ft<sup>3</sup>/s. Figure 39 also shows the path of a boat passing over the main crest.

The model tests indicate that a boat will take approximately 1 minute (prototype) to float from boatchute 1 to boatchute 2 at 500 ft<sup>3</sup>/s. At 1,500 ft<sup>3</sup>/s, it will take about 30 seconds to reach

boatchute 2. It will take about 40 seconds to reach Union Avenue dam from the Union Avenue bridge at 1,000 ft<sup>3</sup>/s.

### **Sediment Studies**

**Sluicing flows.** - Initially, the model was operated for riverflows from 50 to 3,000 ft<sup>3</sup>/s. Sand spilled over the low wall in front of the Englewood intake for various operating conditions. Operating the radial gate and sluiceway moved sand through the intake areas; however, some sand deposited below the level of the intakes. For low flows, the amount of sand deposited depends on the proportion of water being drawn off through the Englewood water intakes compared to the sluiceway flow. When the radial gate was fully opened, sand was cleaned from the intake area. Tests were conducted to determine how much sluicing could be tolerated during riverflows which would be boatable. An open barrier was placed in the model in front of the water intakes to exclude boaters (fig. 14). To determine tolerable sluicing, model boats were placed in front of the intake. As the radial gate opening was increased, the current began drawing the boats toward the barrier. At high gate openings, the model boats were pinned against the barrier by the current. This condition was considered to be unacceptable. Figure 40 gives the boating limits determined during the sluicing tests. During these tests, the flow through the Englewood intake was set at 30 ft<sup>3</sup>/s.

The city of Englewood currently operates at radial gate settings ranging from 2 to 8 feet to sluice sediment for riverflows ranging from 200 to 3,000 ft<sup>3</sup>/s. At 3,000 ft<sup>3</sup>/s, the upstream water intakes must be closed due to sediment buildup, even with the sluice gate fully opened. These sluicing flows would draw boaters to the barrier and pin the boats against it; therefore, the sluicing flows would be unacceptable for boating.

In order to improve sluicing and reduce the risk to boaters, the low wall currently in front of the intakes was increased in height to match the present elevation of the primary intake wall (5296.5 feet). All of the water entering the intake area was forced to enter at the upstream end of the intake. The high wall extends downstream to the crest of the existing dam and has a 1-foot opening, as shown on figure 3, to allow small amounts of debris to exit the sluiceway. In order to assess the effect of this change, velocities were measured in the intake area with and without the high wall in place. Table 2 summarizes the data.

As shown by the data, the sluicing action is greatly enhanced by adding the high wall. The same velocity can be obtained at a 25-percent gate opening with the high wall as with a 100-percent gate opening with a low wall. When the sediment test was conducted with 3,000-ft<sup>3</sup>/s riverflow and a 30-percent gate opening with the high wall, the primary intake area was almost entirely sluiced out; therefore, sluicing is much more effective with the high wall.

The radial gate can be operated automatically (as it is now) to maintain a constant water surface of about 5290 feet. At riverflows of 50 to 150 ft<sup>3</sup>/s, all or most of the flow enters the Englewood intake channel. As the riverflow increases, the radial gate can be adjusted to facilitate sluicing. The model tests indicated that a sluiceway gate opening of about 50 percent would provide maximum sluicing capacity (table 2).

**Approach flow to Englewood Intake.** - With the high wall in place, an open bar barrier or other type of open barrier will be required to prevent boaters from entering the sluice at the upstream end.

A series of tests were conducted to determine the required placement of the barrier. Figures 41 through 43 show velocities measured for various operating conditions, including river discharges of 240, 811, and 1,450 ft<sup>3</sup>/s. Three different barrier locations were considered. The tests show that the angle of the velocity vector with the barrier would pin the boats against the barrier unless the barrier extends upstream parallel to the angle of the Englewood primary intakes (recommended barrier location on the figures). For this position, the magnitude of the velocity component perpendicular to the barrier does not exceed 2.5 ft/s and the component parallel to the barrier is larger. This would tend to move the boats downstream parallel to the barrier rather than pinning them against the barrier.

**Sediment tests.** - A typical bedload particle size distribution curve was obtained from WWE for this section of the South Platte River near Oxford Avenue, located approximately 1 mile downstream of Union Avenue. The particle size distribution curve was scaled based on techniques outlined in the Sediment Scaling section of this report. Estimates of bedload discharge rate were made for a large spring flow of 3,000 ft<sup>3</sup>/s, which has a return period of 10 years (Wright Water Engineers, 1987).

The model riverbed was filled with sediment sized according to the gradation determined from the scaling calculations. The bedform was shaped according to cross sections provided by WWE. Templates were used in the model to form the sand according to the field data. Only the area from the main dam to just upstream from the Union Avenue bridge contained sand.

Sediment discharge was estimated based on a bedload particle size distribution curve for the South Platte River near Oxford Avenue and river characteristics, including top width, mean depth and velocity, water discharge, water surface slope, and water temperature. These data were entered into a computer program to determine bedload sediment discharge rates using several sediment equations, including Schoklitsch, Kalinske, Meyer-Peter and Muller, and Rottmer. Using the discharge scale ratio ( $L_r^{2.5}$ ), the bedload discharge scaled to 70 pounds/hour or an application rate of 17 pounds every 15 minutes in the physical model.

At a flow of 3,000 ft<sup>3</sup>/s (10-year flood), sediment tests were run with a low wall at radial gate openings of 30 and 100 percent (2.5 and 8 feet). At the 30-percent opening, a large deposit formed in the Englewood intake area covering the first three water intakes. The sediment was shallower near the upstream end of the intakes, where the flow enters. At the 100-percent radial gate opening, some of the sediment deposit was reduced. However, the first few water intakes were still covered.

The sediment test at a riverflow of 3,000 ft<sup>3</sup>/s was continued for 3 days with the raised solid wall in place along the Englewood intake. Sediment was fed into the model upstream of the Union Avenue bridge every 15 minutes. After 3 days, the deposition in the pool between the Union Avenue dam and the first rockfill dam appeared to be stable. Figures 44 and 45 are photographs of the pool prior to the sediment test. Figures 46 and 47 show contours in the pool before and after the test.

A large sand bar was deposited downstream of the main dam to the left of the boatchutes almost to the end of the original stilling basin wall (fig. 48). Another deposit formed downstream of the boatchute in an alluvial fan shape (fig. 49). Downstream from the sluice gate a deposit formed in the submerged sluice area; however, there was no indication of any deposition in the area just

downstream of the radial gate. The high velocity under the gate keeps this area clear. The operation of the radial gate and the sluicing capacity are not affected by the downstream tailwater. Figure 50 shows the dunes that formed upstream of the Union Avenue dam.

The deposits indicated by this test would probably take several years to occur, unless there is a relatively large flood in 1 year.

Sediment deposits affect flow patterns in the pool. However, flow patterns are acceptable for boating before or after deposition occurs. Before the sand deposits, the flow gradually turns toward the second chute on the left bank. A back eddy forms downstream from the dam which will carry boaters or sediment back towards the toe of the upstream dam. After the pool partially fills, the back eddy is eliminated and the flow forms a channel turning gradually from boatchute 1 to boatchute 2.

### **Floodflows**

Discharges up to the COE 100-year flood of 16,400 ft<sup>3</sup>/s were observed in the model. Figure 51 shows the model operating at the 100-year floodflow. Table 3 lists the water surface profiles measured at flows ranging from 500 to 16,400 ft<sup>3</sup>/s. Model data indicate that the water level is contained within the streambanks up to the 100-year flow. Profiles through boatchute 1 are plotted on figure 52. Profiles were measured down the centerline of boatchute 1. The wave was highest along the centerline due to the V-pattern downstream of the boatchute. However, the height of the wave did not cause a major problem for the model boats navigating the boatchute. It was found that the height of the wave could be reduced by spreading the wave across the width of the boatchute; however, this was not considered to be desirable since the boats turned sideways when the wave was spread out.

The undular shape of the wave downstream of the boatchute was maintained throughout the entire flow range. Undesirable reverse rollers, which may trap boaters, do not form at any flow.

**Flow over the main crest.** - At flows greater than 200 ft<sup>3</sup>/s, the water started overtopping the main spillway crest. The exact amount of flow will depend on the amount of sluicing and flow into the Englewood intake. For flows between 200 and 1,000 ft<sup>3</sup>/s, boats tended to "hang up" on the main crest and downstream wedge (fig. 18) due to shallow water. When the water was deep enough for boats to clear the crest, they passed on through into the tailwater pool area. The configuration with the wedge and downstream riprap prevented reverse rollers from forming downstream from the main crest. The 3-foot riprap on the downstream face of the dam experienced only minor movement during the 100-year flood event; therefore, 3 feet of riprap should be adequate to protect the embankment. The area between the boatchute and the sluice wall experienced major erosion during the 100-year flood; therefore, some type of stabilization is recommended for this area.

During the model tests, the size of the rockfill embankment was reduced by shortening the 1 to 10 slope to 130 feet long, before dropping off at a 3 to 1 slope to the bottom of the stilling basin (fig. 53). This change reduced the amount of material required by 1,500 yd<sup>3</sup>. The additional material was not necessary since the area below the tailwater level to the left of boatchute 1 will be a deposition area. A large eddy formed in the pool which caused a large sand deposit where the rockfill embankment was originally planned. The sand bar accumulation continued to almost the end of the original stilling basin wall. When a 100-year flood was observed after the sand bar

was deposited, some of the sand moved further downstream into the pool between boatchute 1 and boatchute 2.

**Lowering the sluice wall.** - In the original boatchute design constructed in the physical model, the sluice wall adjacent to boatchute 1 was raised to elevation 5291. This elevation was the same as the east sluice wall along the South Platte riverbank. The wall was raised because the 10 to 1 slope of the rock dam extended to the end of stilling basin wall (fig. 13). When the length of the 10 to 1 slope was shortened, as described above, the wall was lowered to the original elevation of 5285 feet (fig. 29). The original wall elevation worked well for all boating flows (100 to 3,000 ft<sup>3</sup>/s) tested. If a boater entered the sluiceway and floated upstream close to the radial gate, he/she could be trapped next to the radial gate. A barricade above the minimum water surface would be appropriate to prevent boaters from entering the radial gate area.

Not raising the sluice wall will reduce construction costs since raising the sluice wall and reinforcing it to withstand differential loading forces would have been costly.

**Riprap and channel stability.** - The riprap on the first rock embankment downstream of the Union Avenue dam was stable for all flows studied. The upstream face of the embankment was mainly a deposition area. After the sand reached the elevation of the crest (5288 feet), it passed over the embankment into the next pool. Riprap on the downstream face was stable through the 100-year flood.

The orientation of the embankment was modified during the tests to minimize the possibility of downstream channel erosion on the left or right bank. The orientation of boatchute 2 was changed to align the direction of the flow to be parallel to the streambank. A wedge-shaped area was added to the top of the embankment on the right to direct the flow away from the right bank of the river. Both modifications had a positive effect on the flow downstream of the embankment. The tendency for the flow to attack the banks was reduced. Bank erosion and channel maintenance should be minimized by this design.

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**Table 1. - Average monthly South Platte riverflows at Littleton, Colorado, and average Englewood diversions\***

Month	Average riverflows (ft <sup>3</sup> /s)			Englewood diversions*** (ft <sup>3</sup> /s) 1986
	1982-85 (wet period)	1987**	1972-81	
May	1,300	2,600	700	17
June	1,000	1,200	600	20
July	700	400	400	22
August	700	300	200	25
September	300	100	100	15

\* Personal communication, Bob Ferguson, WWE, 1988.

\*\* Maximum discharge 1982-1987, 3,700 ft<sup>3</sup>/s, May 27, 1987.

\*\*\* Englewood diversions water right is 70 ft<sup>3</sup>/s, operational flow 100 ft<sup>3</sup>/s.

**Table 2. - Velocity comparison in front of primary Englewood water intake with high wall and low wall  
(Q = 260 ft<sup>3</sup>/s) (Q<sub>Englewood</sub> = 30 ft<sup>3</sup>/s)**

Radial gate open		Velocities (ft/s) Point 1*	
(ft)	(percent)	High wall	Low wall
0.5	6	1.8	1.1
1.0	12	2.5	1.4
2.0	25	4.7	1.7
3.0	38	5.1	2.2
4.0	50	8.1	3.7
5.0	63	8.3	4.4
6.0	75	8.3	4.7
8.0	100	8.6	4.7

\* Readings were taken 0.5 foot off the bottom, 38 feet upstream from the sluice gate.

**Table 3. - Water surface elevations from Union Avenue bridge to the pool downstream of boatchute 2**

Distance from bridge (ft)	River discharge (ft <sup>3</sup> /s)					Bottom elevation (ft)
	500	1,000	1,500	5,000	16,400	
	Sluice flow (ft <sup>3</sup> /s)					
	145	320	470	530	1,150	
0.0**	5,290.88	5,291.81	5,292.45	5,295.08	5,300.69	5286.82
229.0	5,290.79	5,291.59	5,292.11	5,294.89	5,299.33	5288.40
252.0	*	5,291.57	5,292.06	*	*	*
261.0	5,290.73	5,291.57	5,292.11	5,294.81	5,300.01	*
265.5	5,290.73	5,291.57	5,292.11	5,294.73	5,299.80	*
270.0	5,290.72	5,291.50	5,292.05	5,294.61	5,298.78	*
274.5	5,290.57	5,291.34	5,291.85	5,294.34	5,298.20	*
279.0***	5,290.28	5,290.84	5,291.30	5,293.68	5,297.96	5288.03
283.5	5,289.54	5,290.13	5,290.54	5,292.83	5,297.17	5287.65
288.0	5,288.95	5,289.51	5,289.92	5,291.89	5,296.15	5287.42
297.0	5,288.17	5,288.59	5,288.92	5,290.45	5,293.99	5286.87
306.0	5,287.36	5,287.78	5,288.06	5,289.25	5,292.24	5286.35
315.0	5,287.42	5,288.27	5,287.85	5,288.84	5,291.18	5285.89
324.0	5,288.73	5,289.41	5,288.95	5,290.79	5,290.91	5286.12
333.0	5,287.53	5,287.91	5,288.27	5,291.63	5,291.88	5283.12
337.8	*	*	5,288.69	*	*	*
342.0	5,287.61	5,289.58	5,290.22	5,291.04	5,293.98	5280.12
351.0	5,288.22	5,288.49	5,288.76	5,290.16	5,294.96	5277.12
412.0****	5,287.77	5,288.59	5,288.95	*	*	*
747.0*****	5,284.52	5,285.83	5,286.23	*	*	*

\* Data not available. Englewood Intake was set at 30 ft<sup>3</sup>/s.

\*\* Downstream side of Union Avenue bridge, 43 feet from the right abutment.

\*\*\* Crest of Union Avenue dam, center of boatchute 2.

\*\*\*\* Pool elevation between boatchute 1 and boatchute 2.

\*\*\*\*\* Pool elevation downstream of boatchute 2.

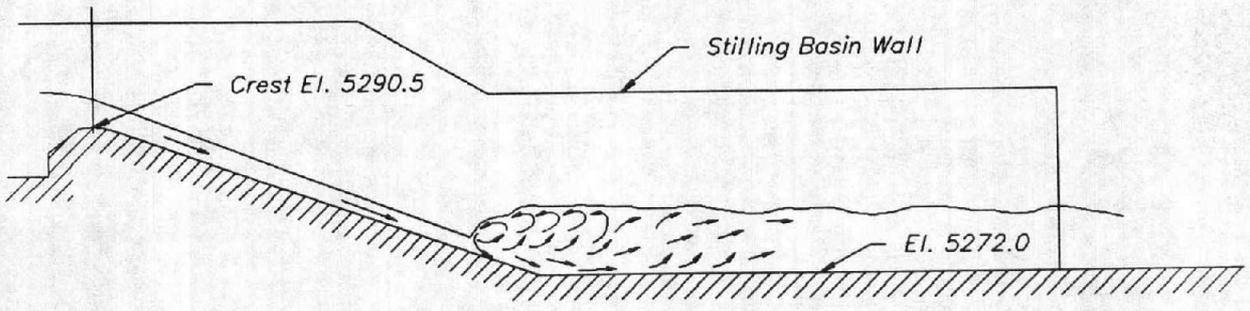


Figure 1. - Section through existing Union Avenue dam showing typical reverse roller hydraulic jump.

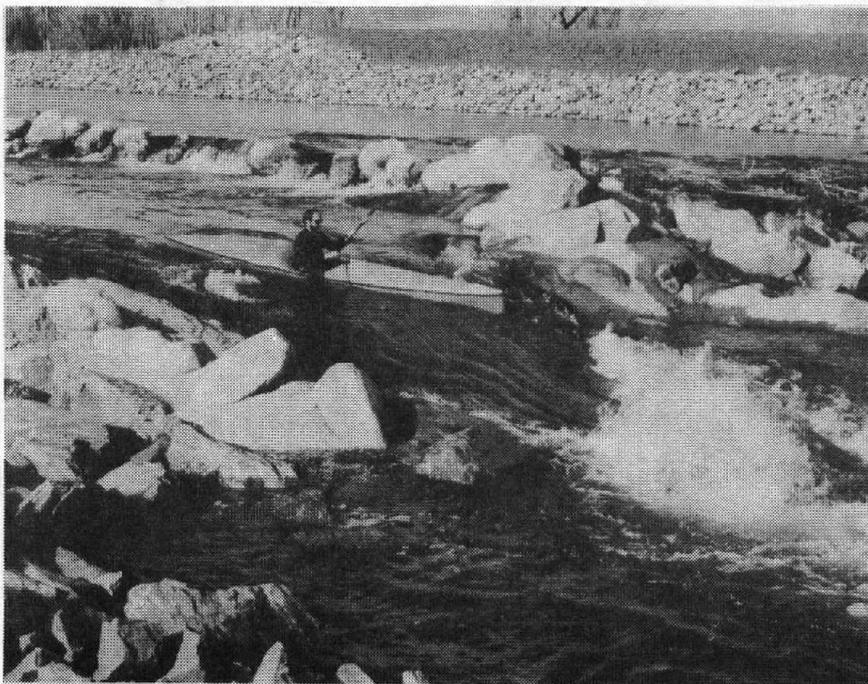


Figure 2. - Kayaker at Brown's Ditch weir boatchute.

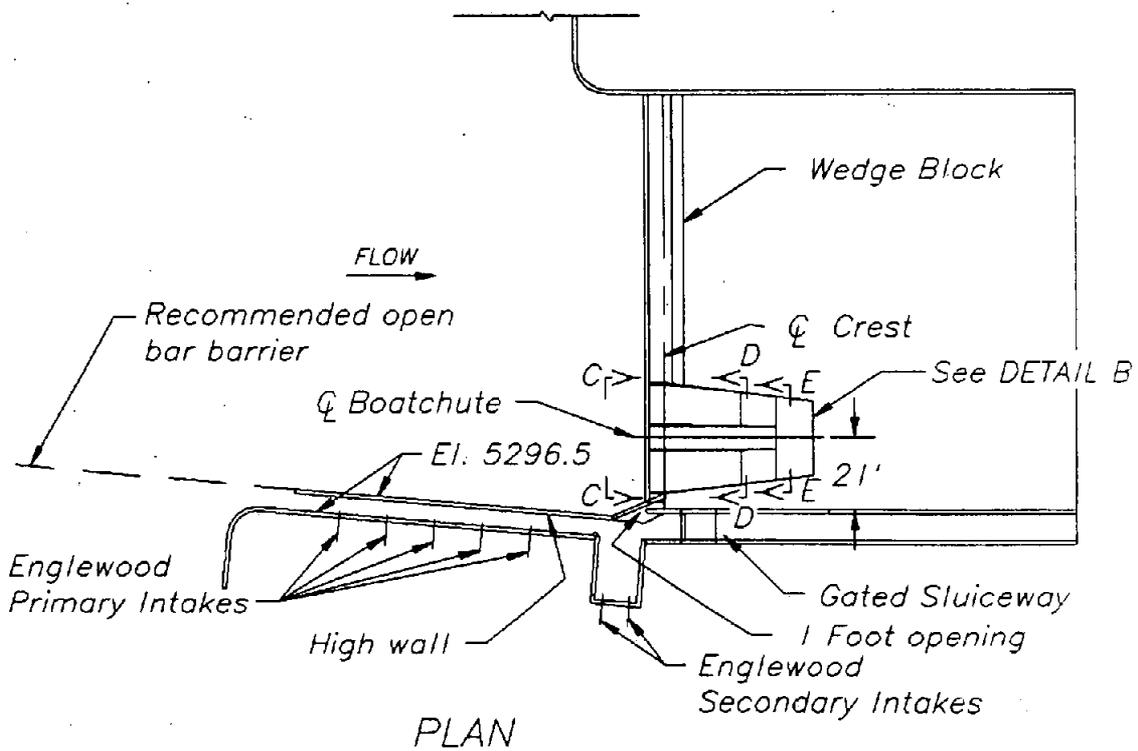
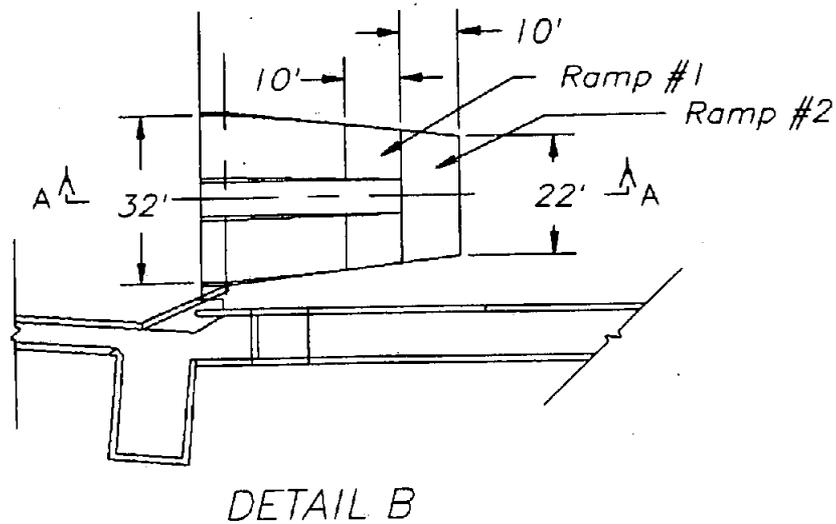


Figure 3. - Recommended boatchute configuration.

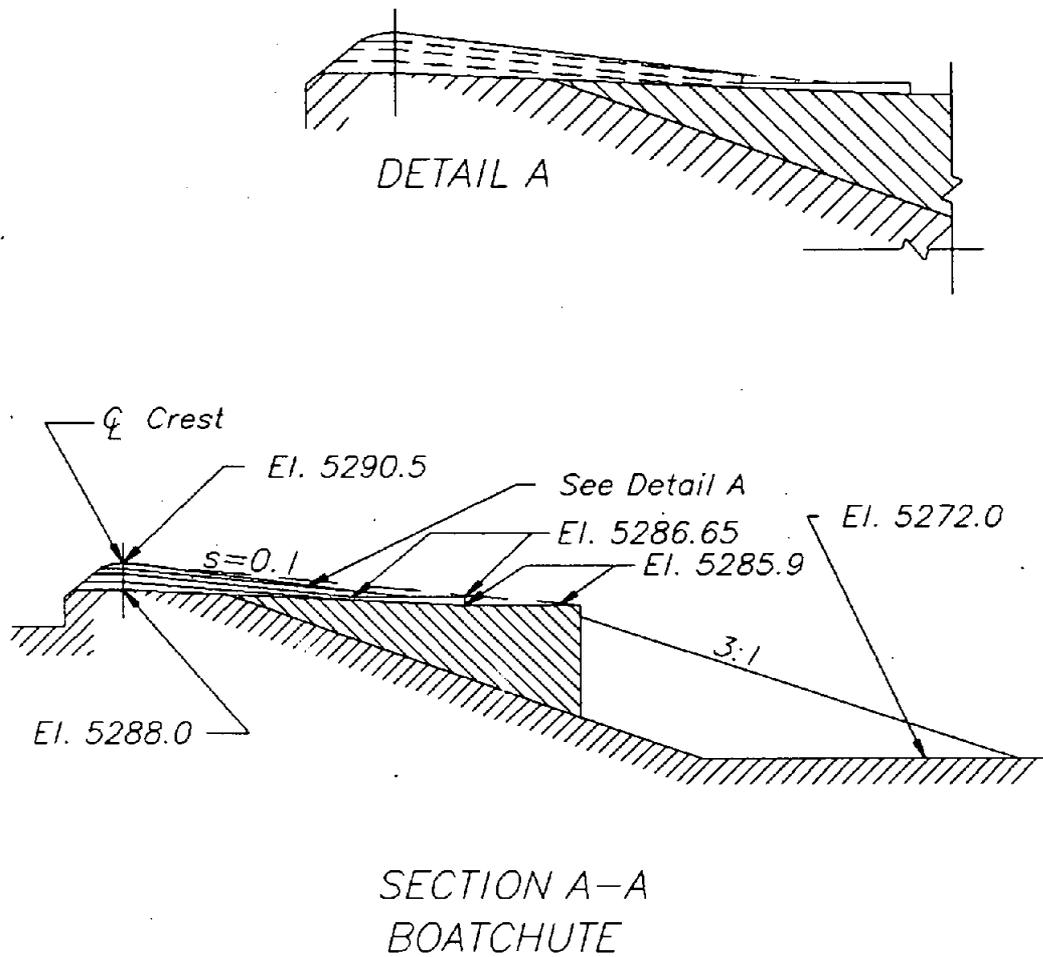
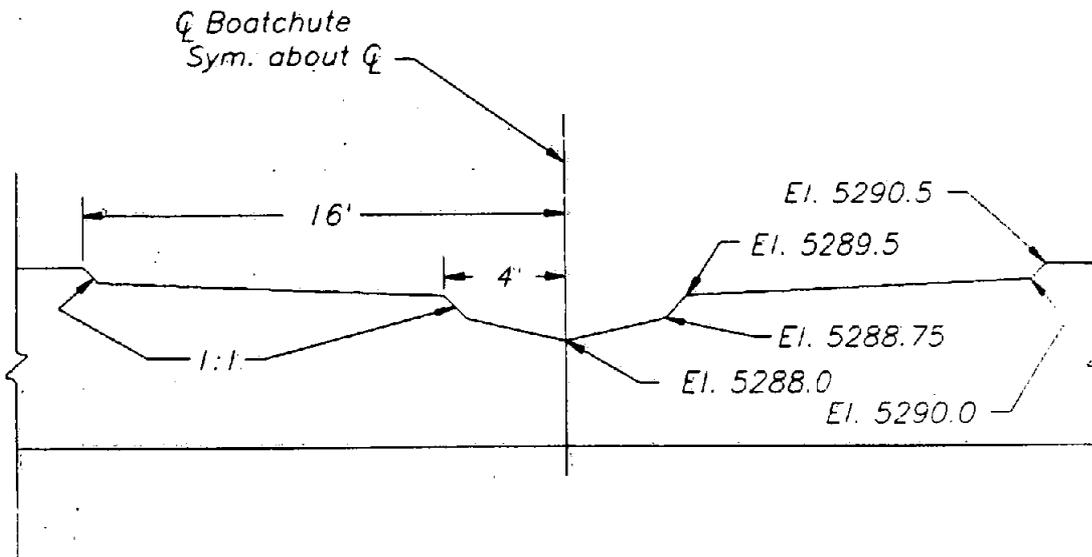
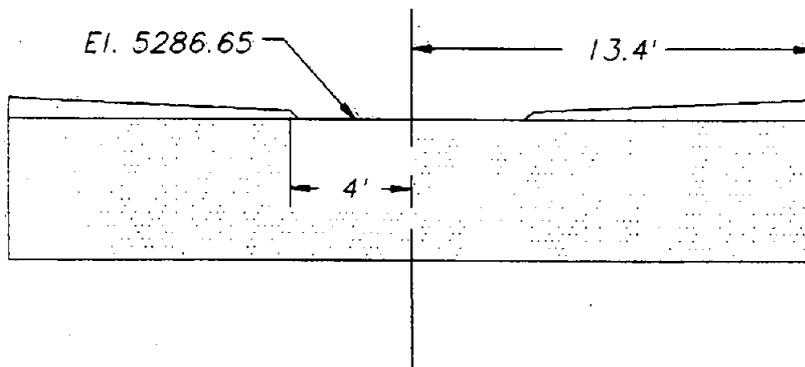


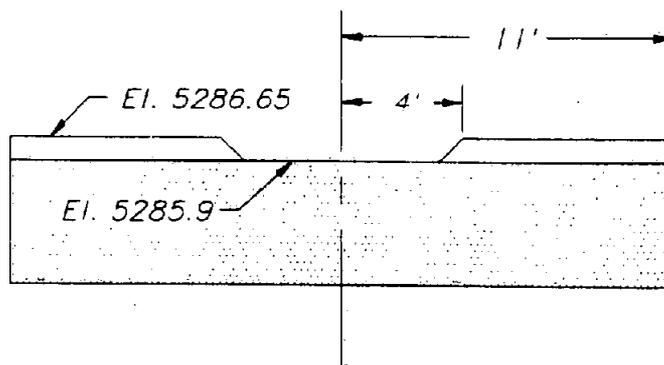
Figure 4. - Section along centerline of Union Avenue dam boatchute (boatchute 1).



VIEW C-C



SECTION D-D



SECTION E-E

Figure 5. - Sections through Union Avenue dam boat chute. (See figure 3 for section locations.)

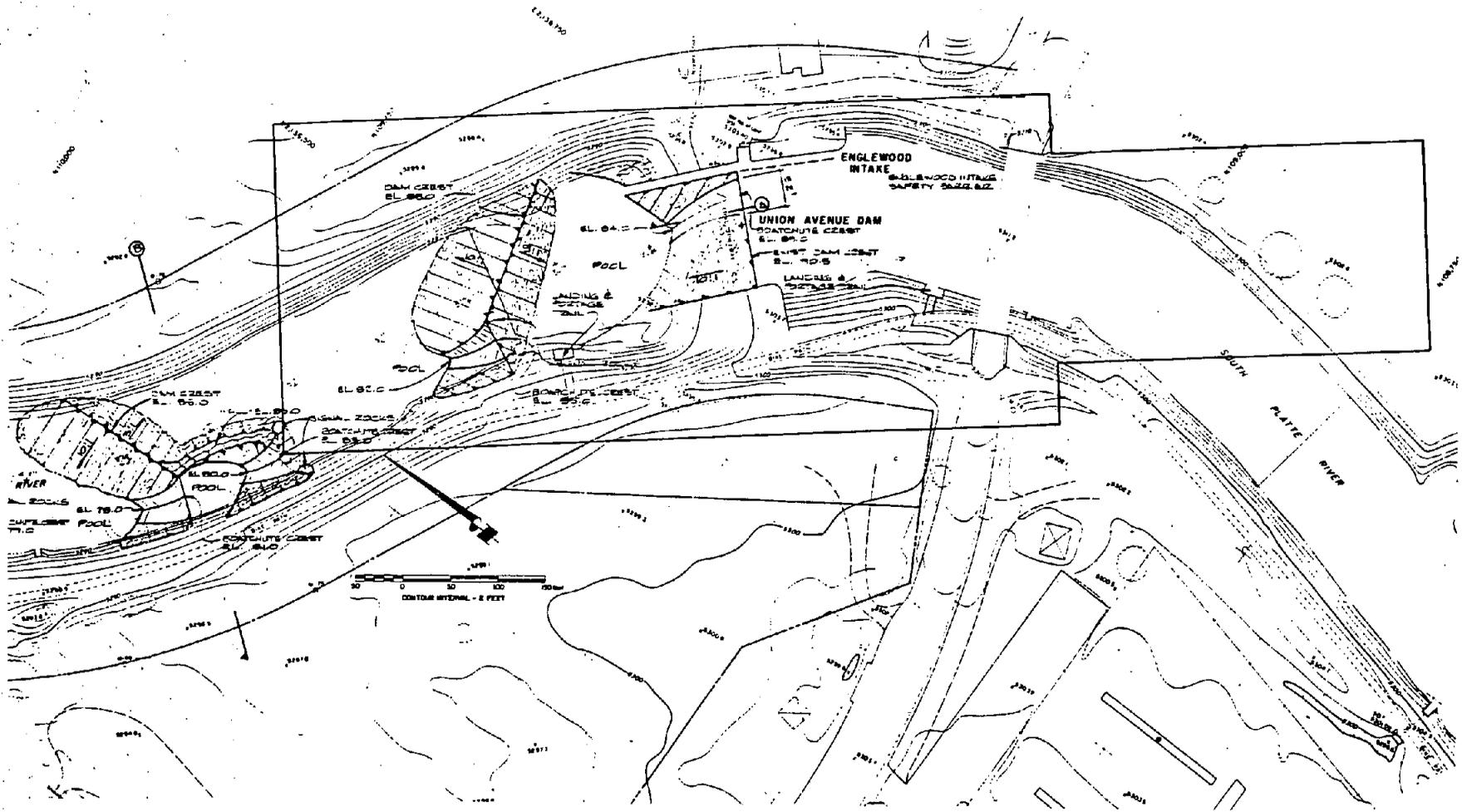


Figure 6. - Model study area.

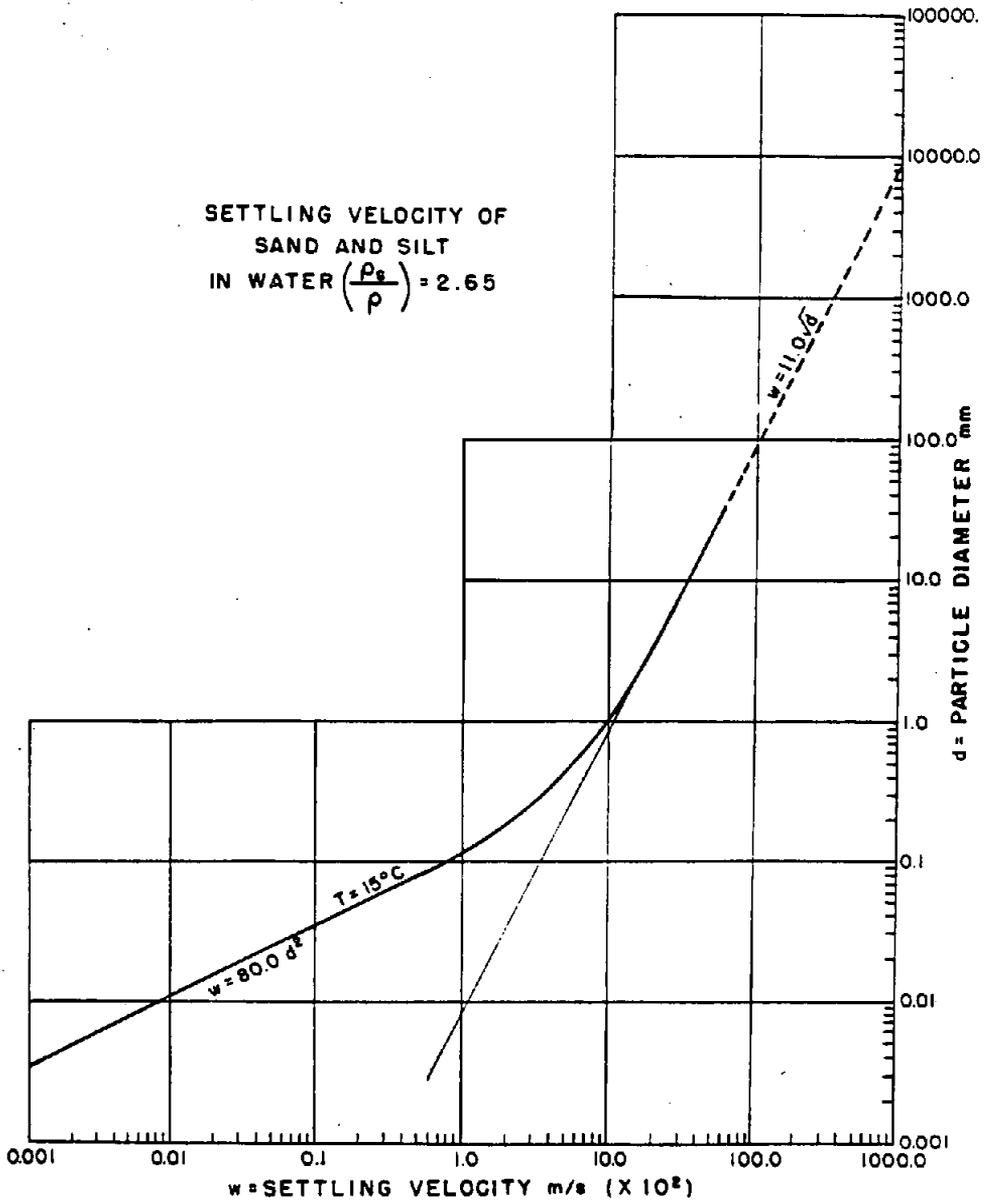


Figure 7. - Settling velocity of sand and silt in water.

## SOUTH PLATTE BOATCHUTE STUDY SEDIMENT SCALING

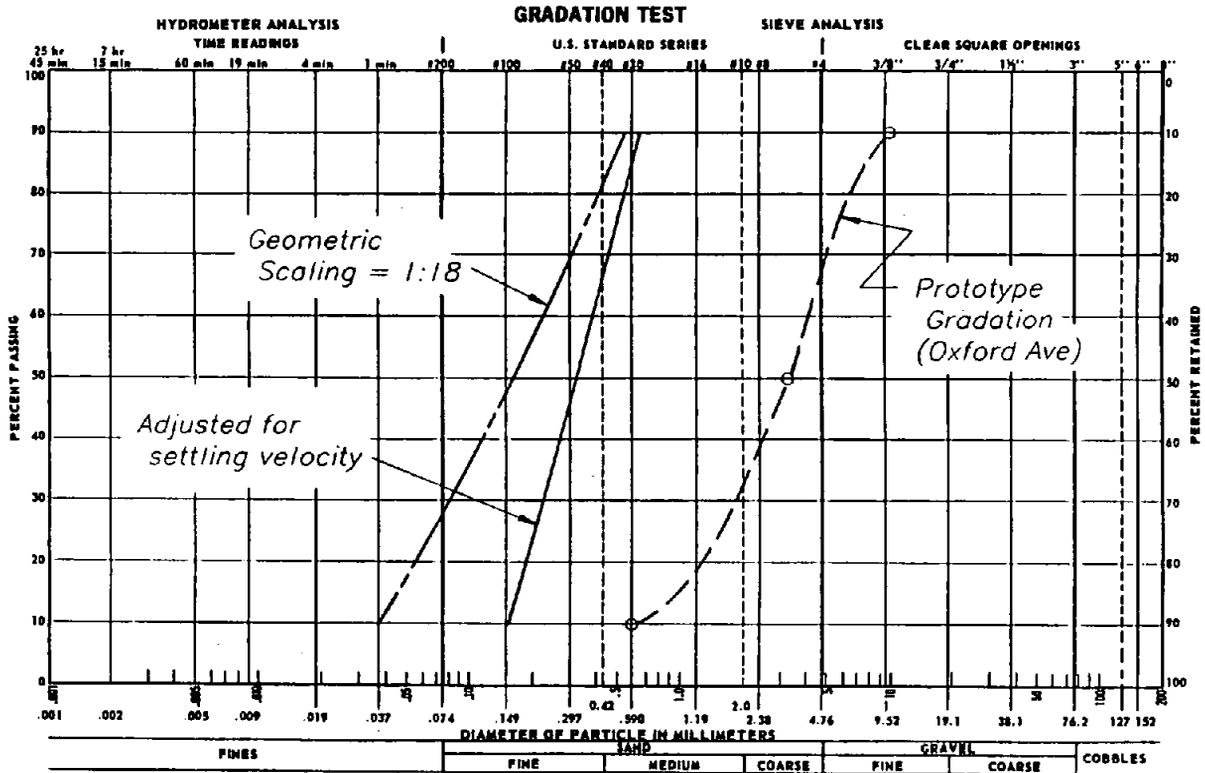


Figure 8. - Model and prototype grain size distribution for bedload.

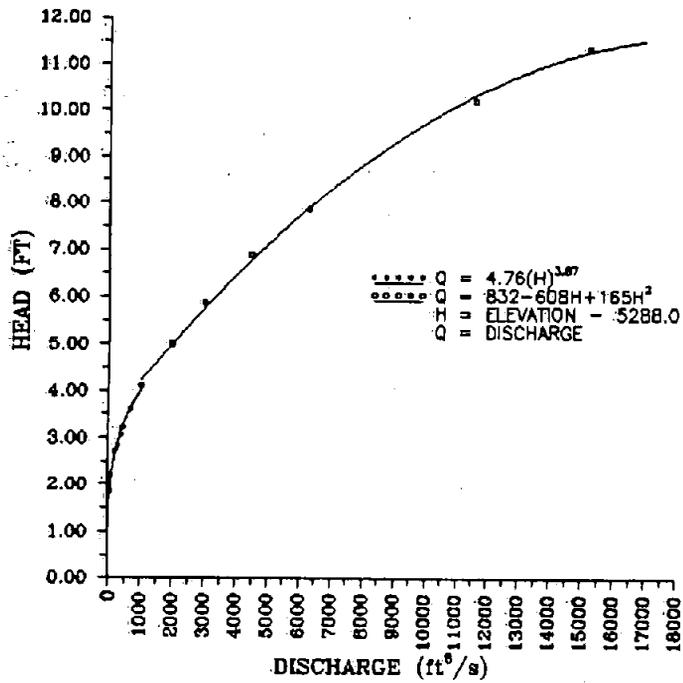


Figure 9A. - Union Avenue dam discharge rating curve (with boatchute).

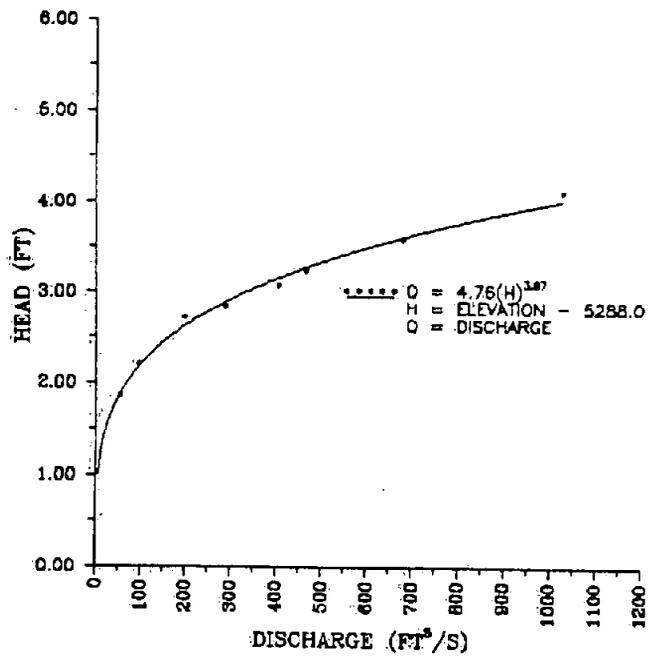


Figure 9B. - Union Avenue dam discharge rating curve (with boatchute).

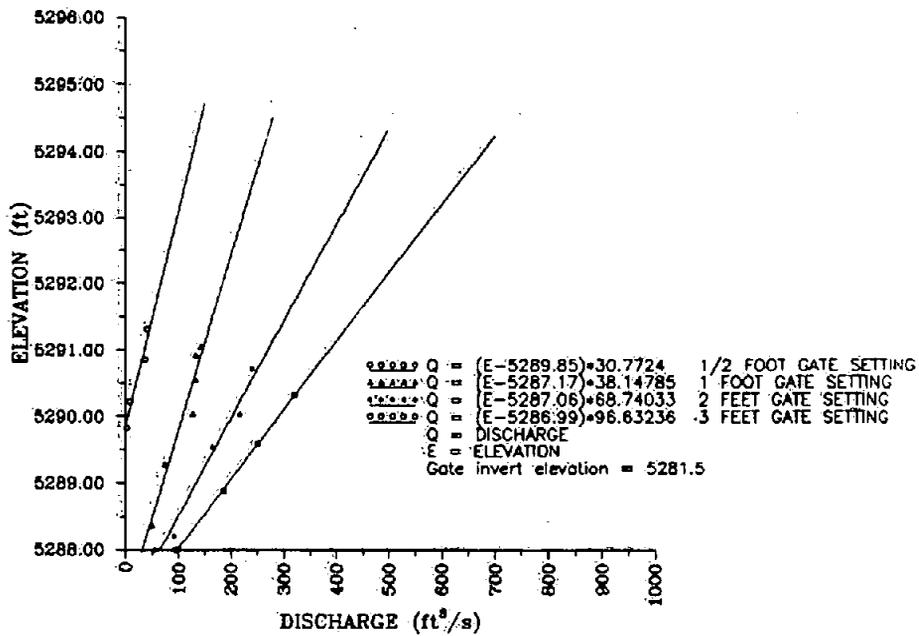


Figure 10. - Union Avenue dam radial gate discharge rating curves with boat chute.

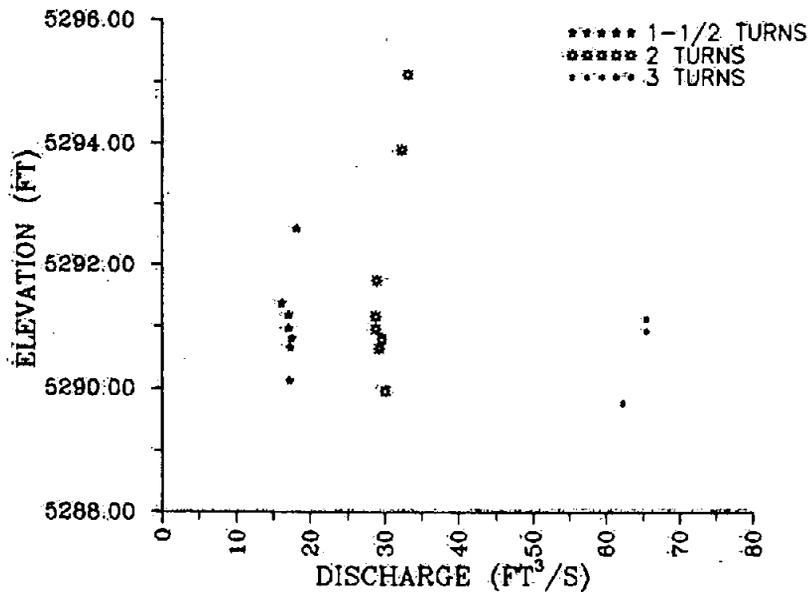


Figure 11. - Englewood Intake model discharge curves.

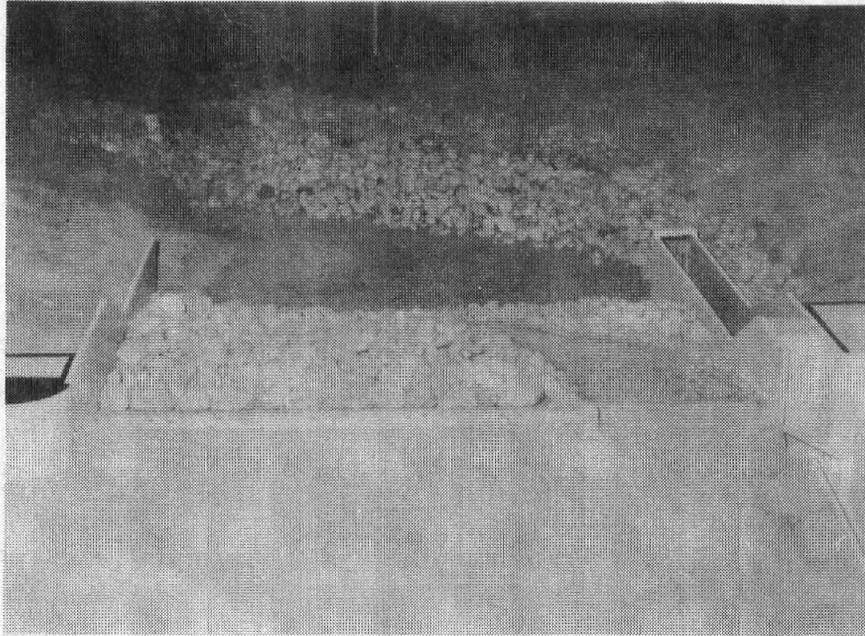


Figure 12. - Boatchute 1 constructed in model looking downstream - original design.

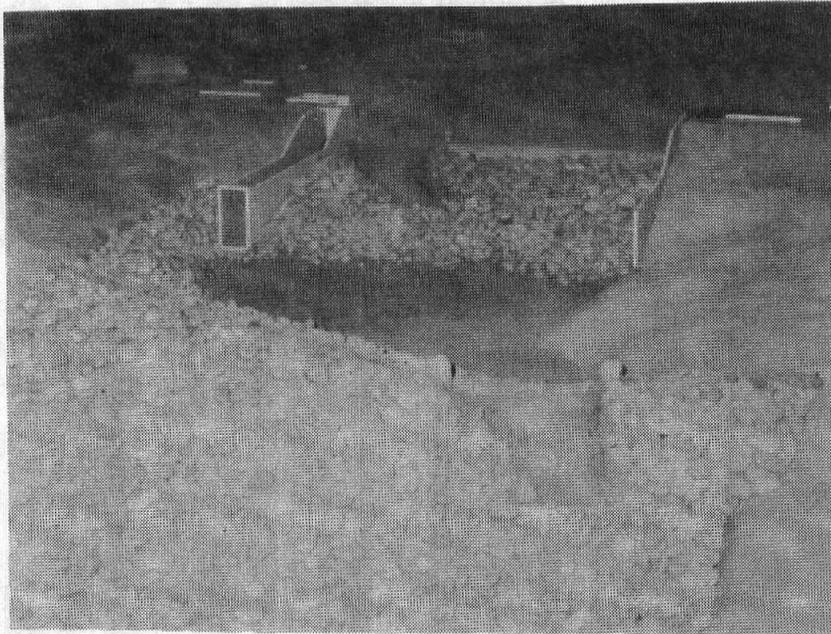


Figure 13. - Boatchute 2 looking upstream - original design.

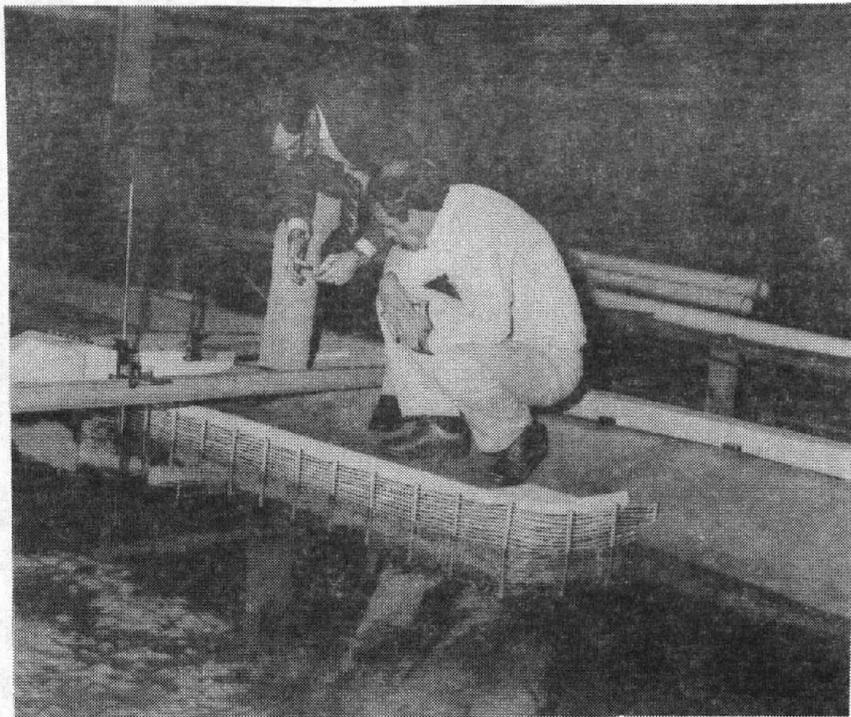


Figure 14. - Open bar-type barricade constructed along Englewood intake.

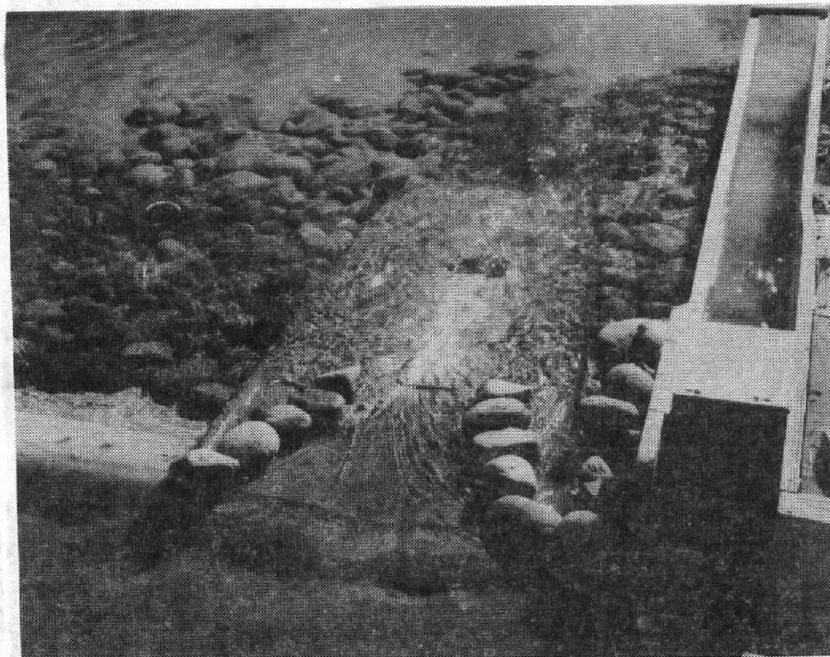


Figure 15. - Union Avenue dam boatchute 1 narrowed with rocks.

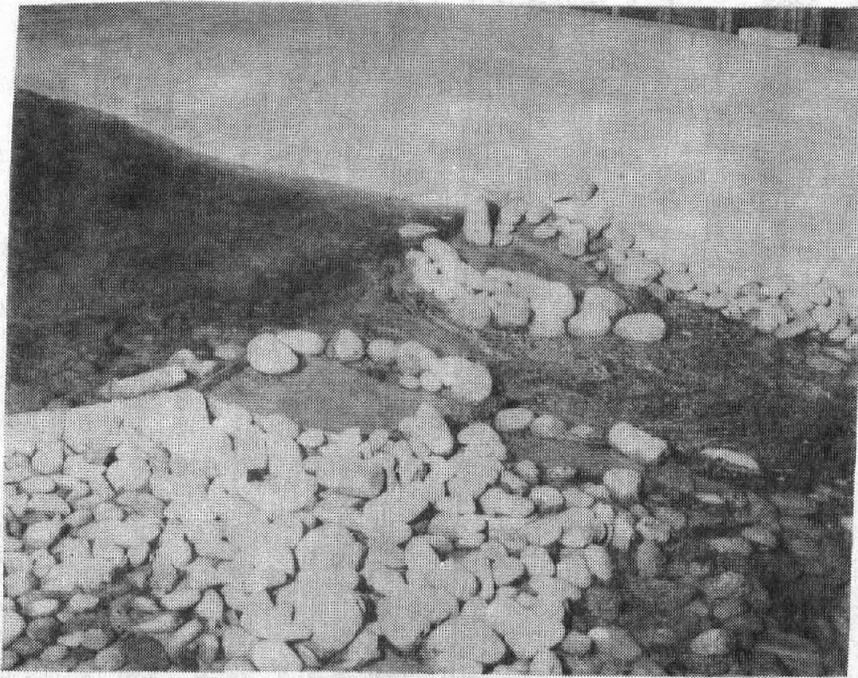


Figure 16. - Boatchute 2 narrowed with rocks.

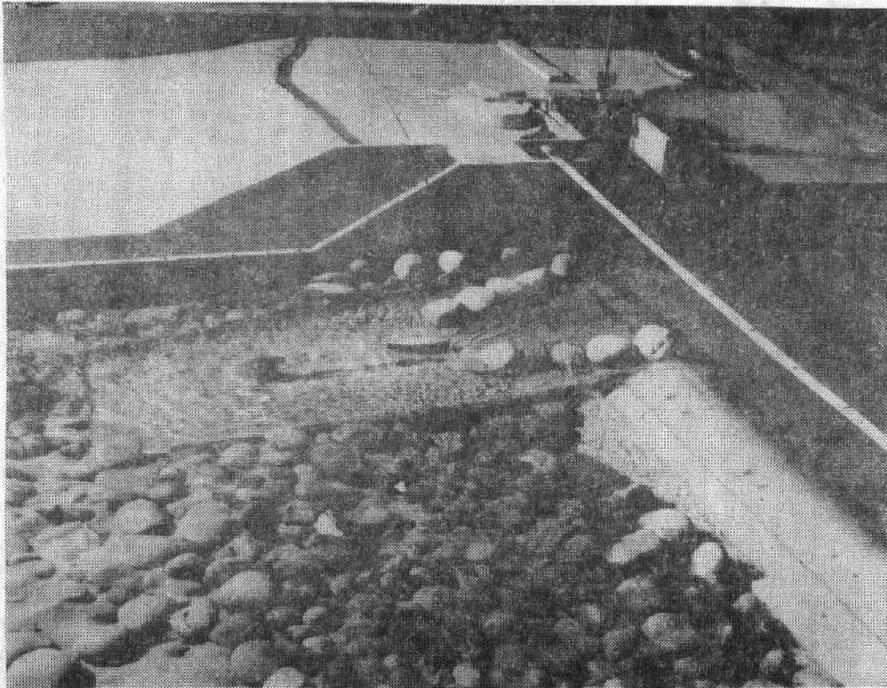


Figure 17. - Modification of Union Avenue dam boatchute with single ramp in center of chute.

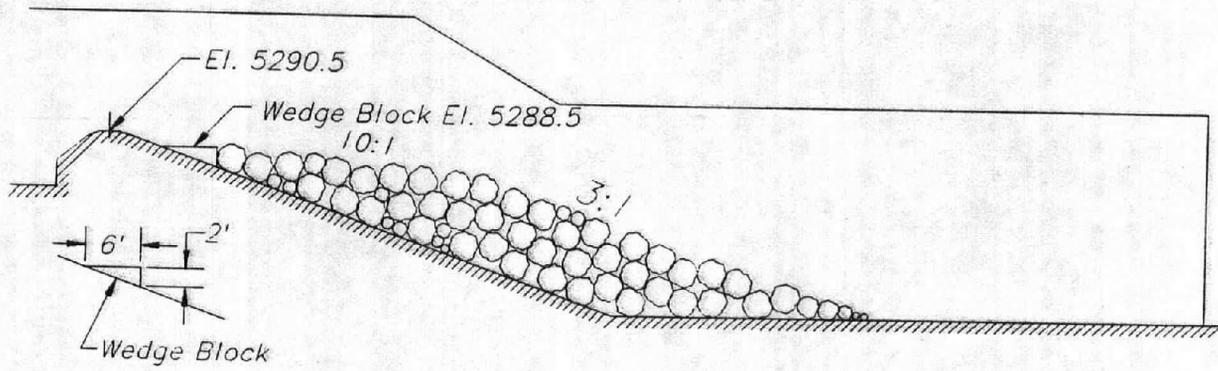


Figure 18. - Section through existing spillway showing modifications with wedge block and rockfill on downstream face.

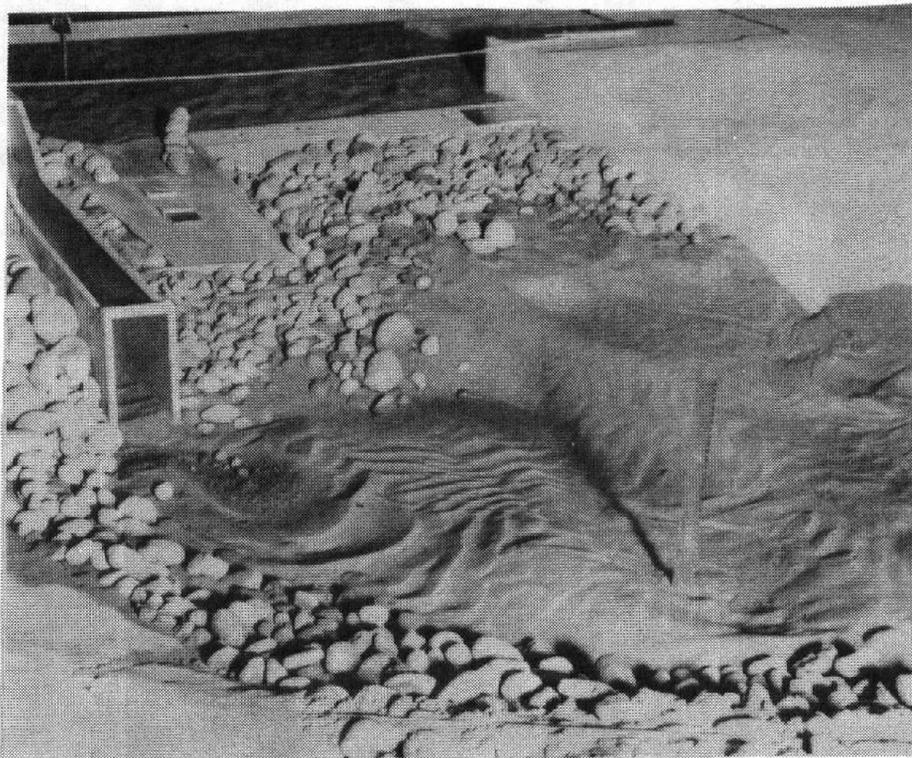


Figure 19. - Design with two ramps in center of boatc chute 1.

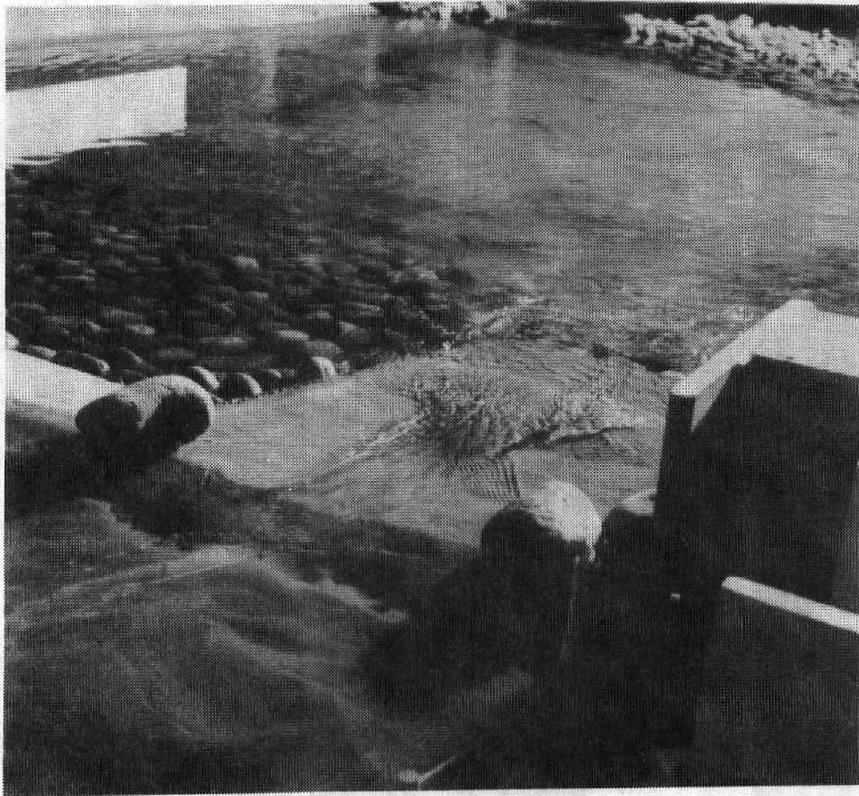


Figure 20. - Boatchute 1 with ramp extending across boatchute, no center trough.

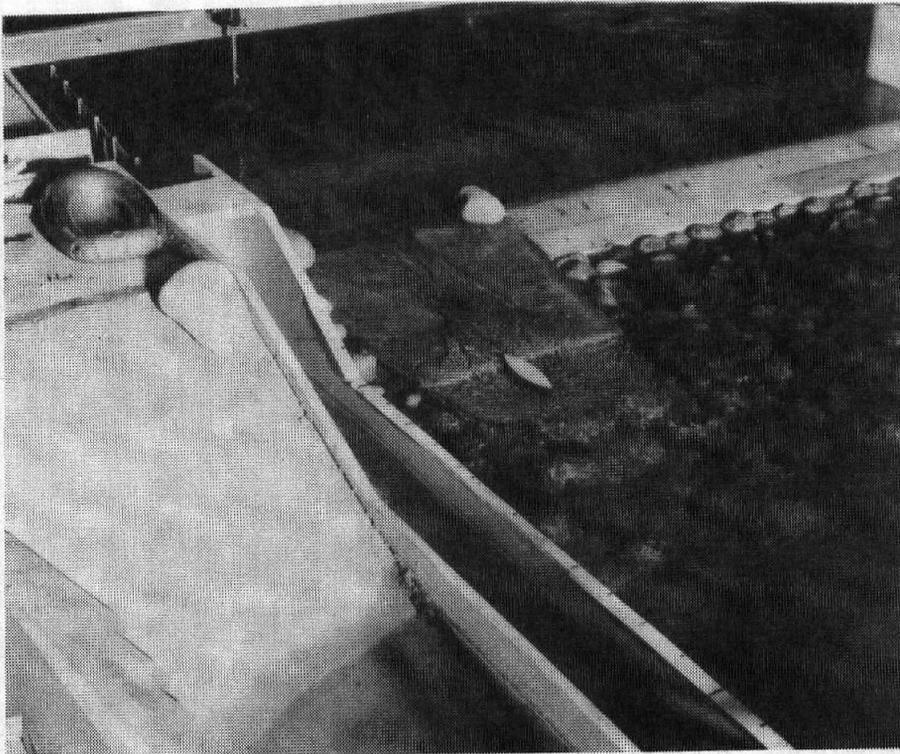


Figure 21. - Kayak proceeding through boatchute.

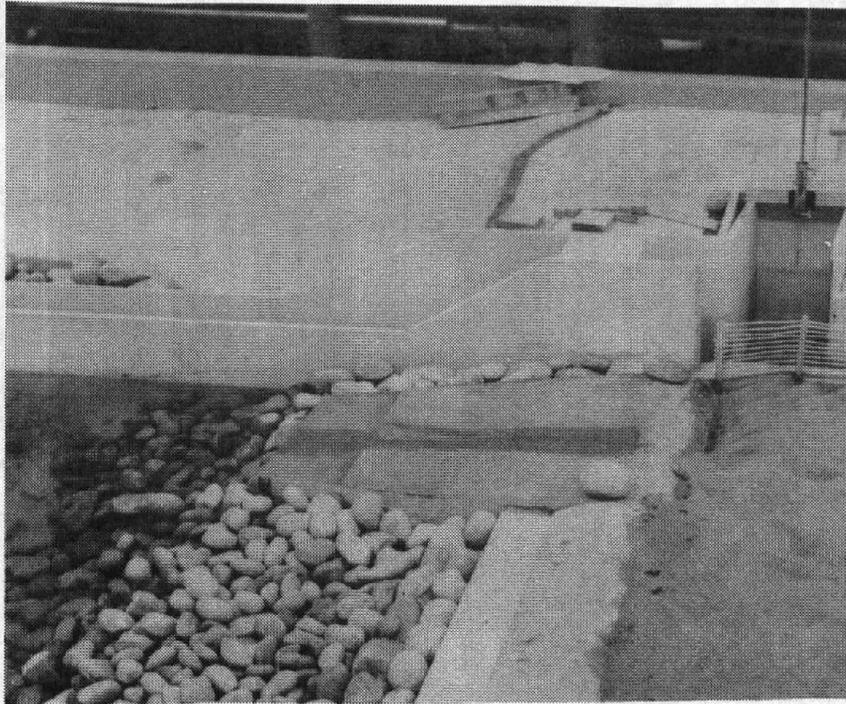


Figure 22. - Boatchute 1 with trough through first ramp.

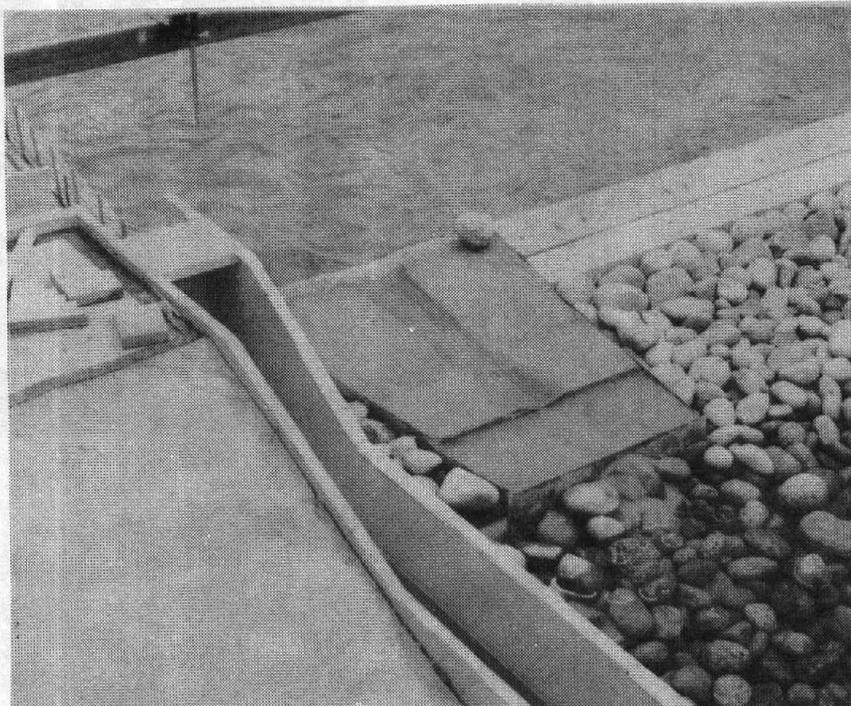


Figure 23. - Another view of boatchute 1.

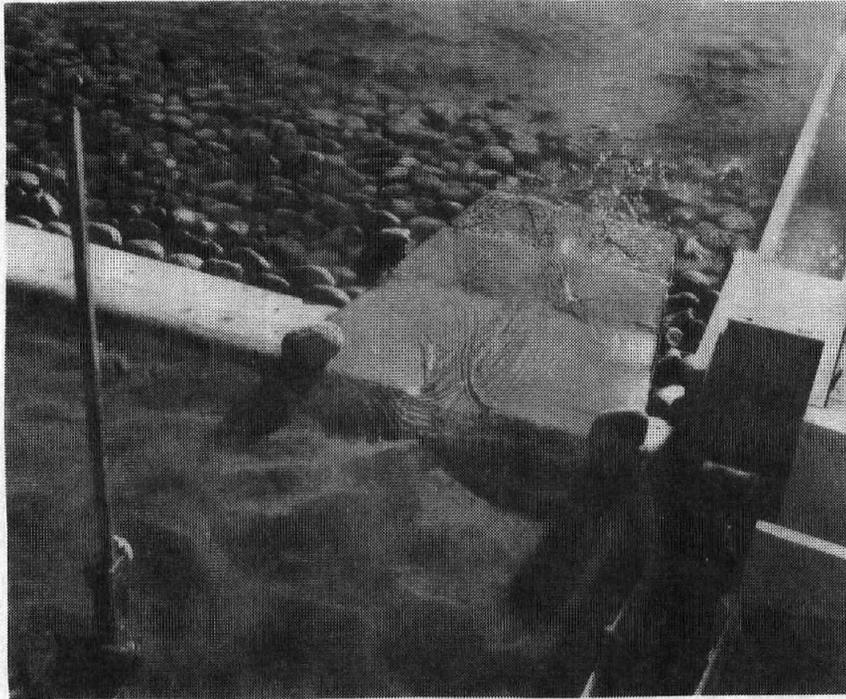


Figure 24. - V-wave pattern in boatchute 1.

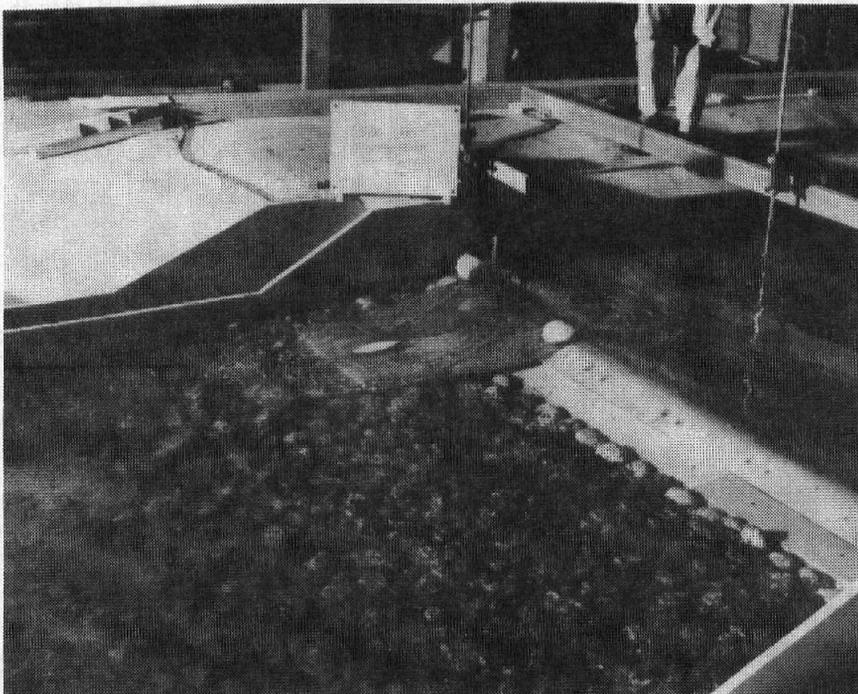


Figure 25. - Boat proceeding through boatchute 1.

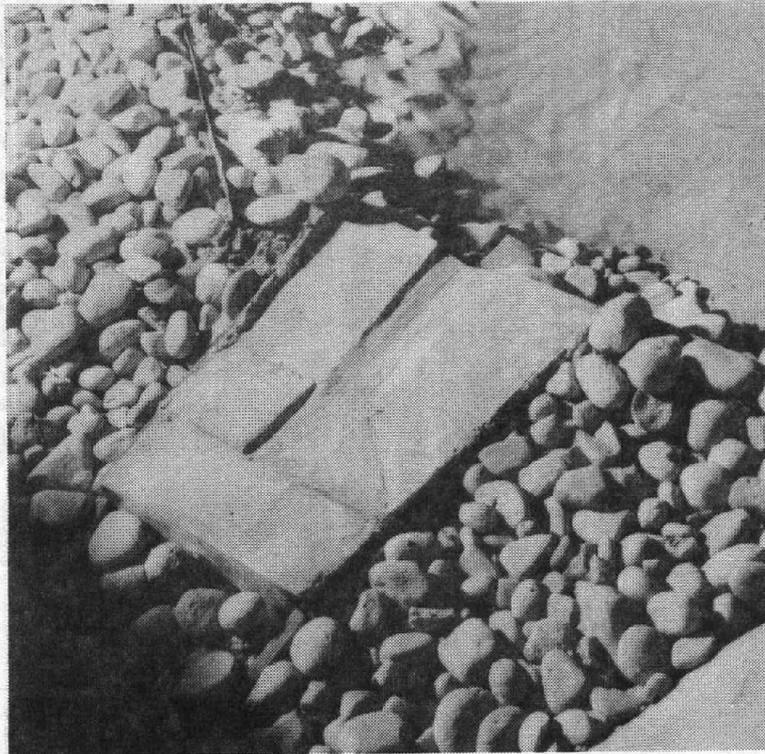


Figure 26. - Boat chute 2.

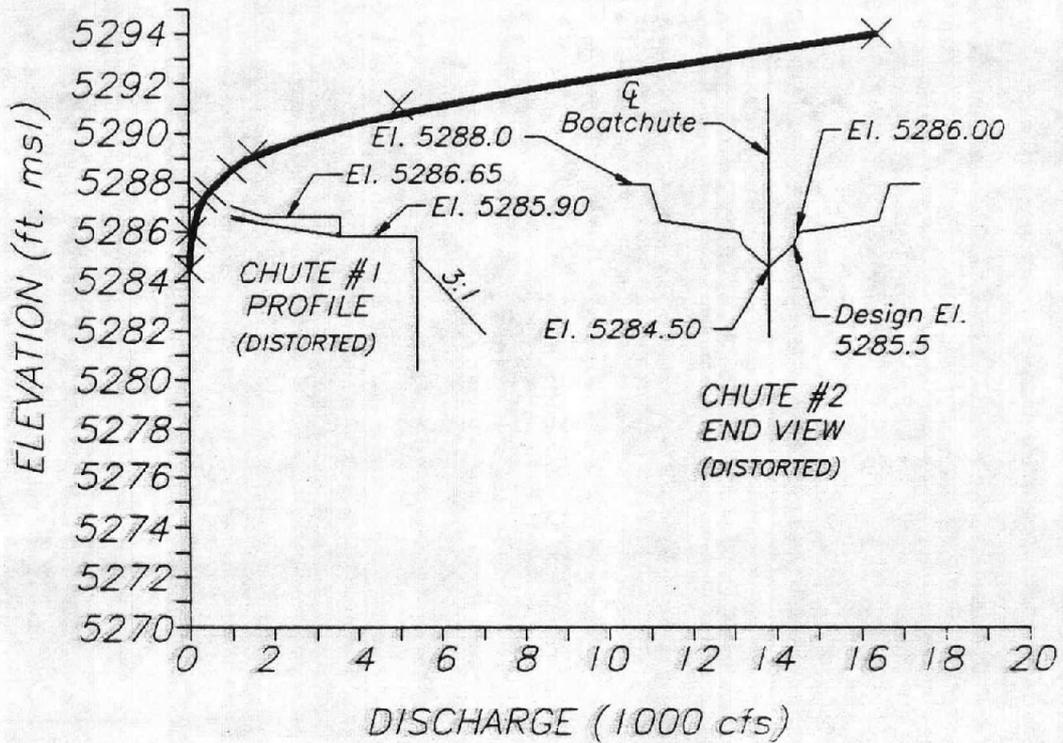


Figure 27. - Water surface vs. discharge in pool between boatchutes 1 and 2.

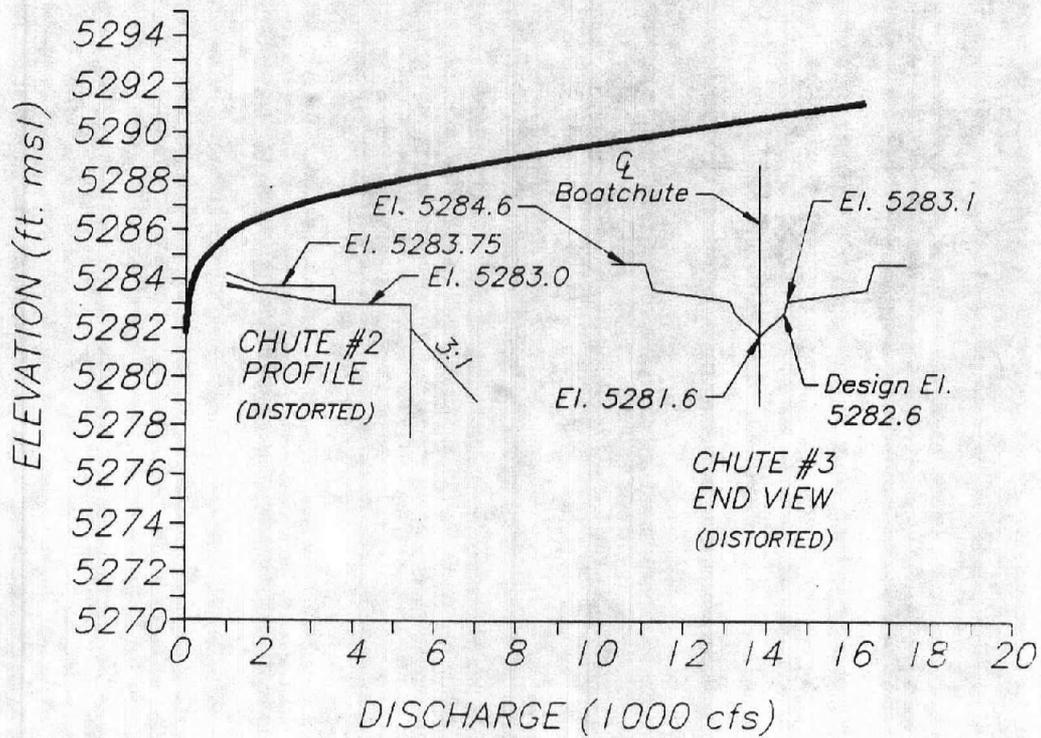


Figure 28. - Water surface elevation vs. discharge in pool between boatchutes 2 and 3.

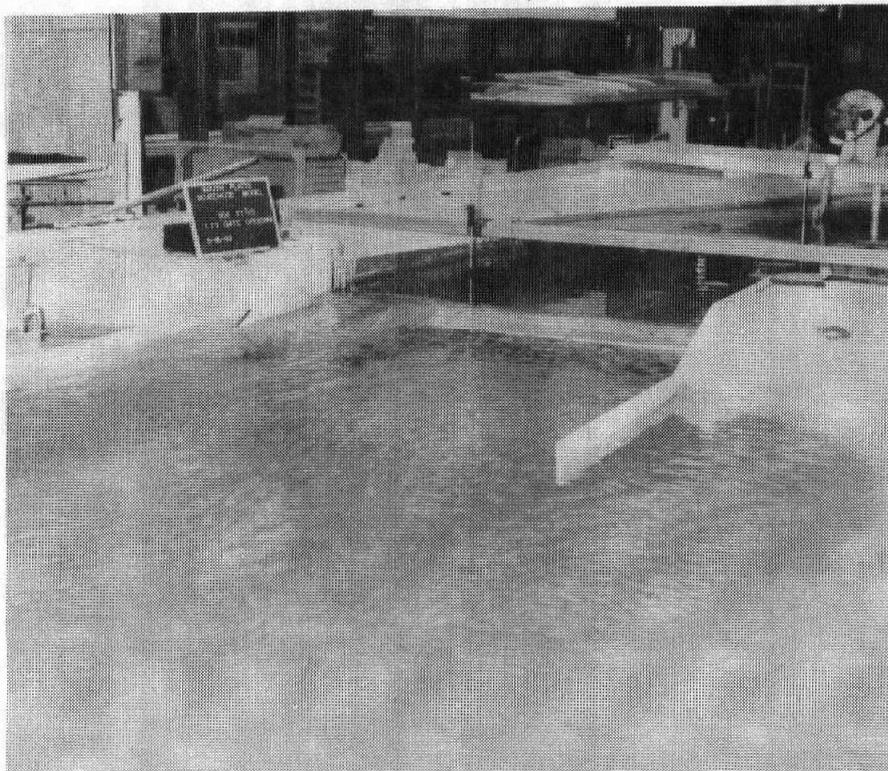


Figure 29. - Overall view of model with final design at 500 ft<sup>3</sup>/s. (Note that left sluice wall has been lowered from El. 5291 to 5285.)



Figure 30. - Boatchute 1 at 500 ft<sup>3</sup>/s (final design).

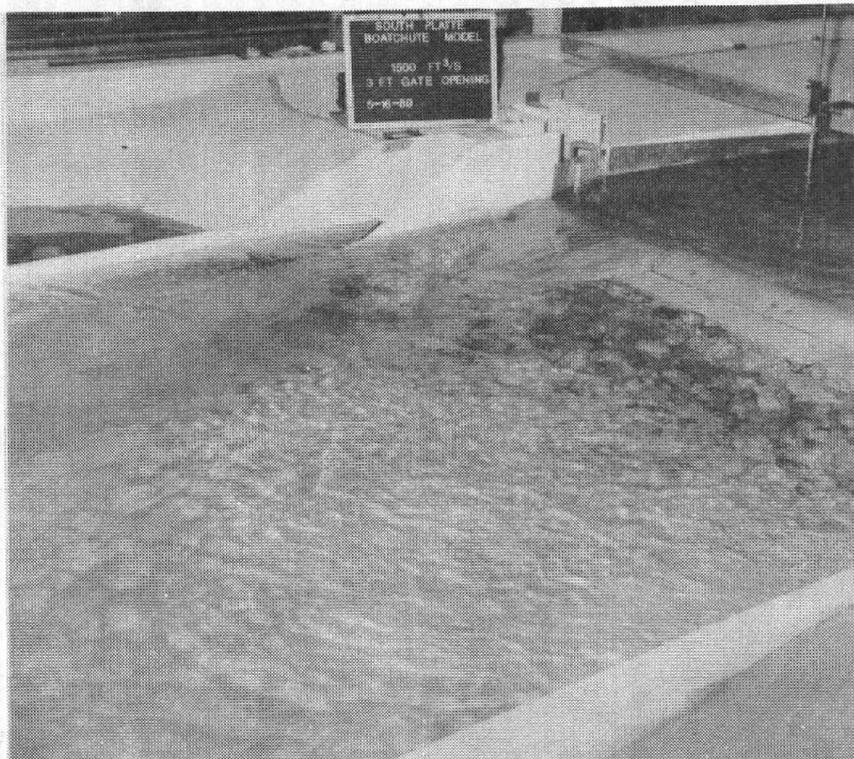


Figure 31. - Boatchute 1 at 1,500 ft<sup>3</sup>/s (final design).



Figure 32. - Boatchute 2 at 500 ft<sup>3</sup>/s (final design).



Figure 33. - Boat going through boatchute 2 at 1,500 ft<sup>3</sup>/s (final design).

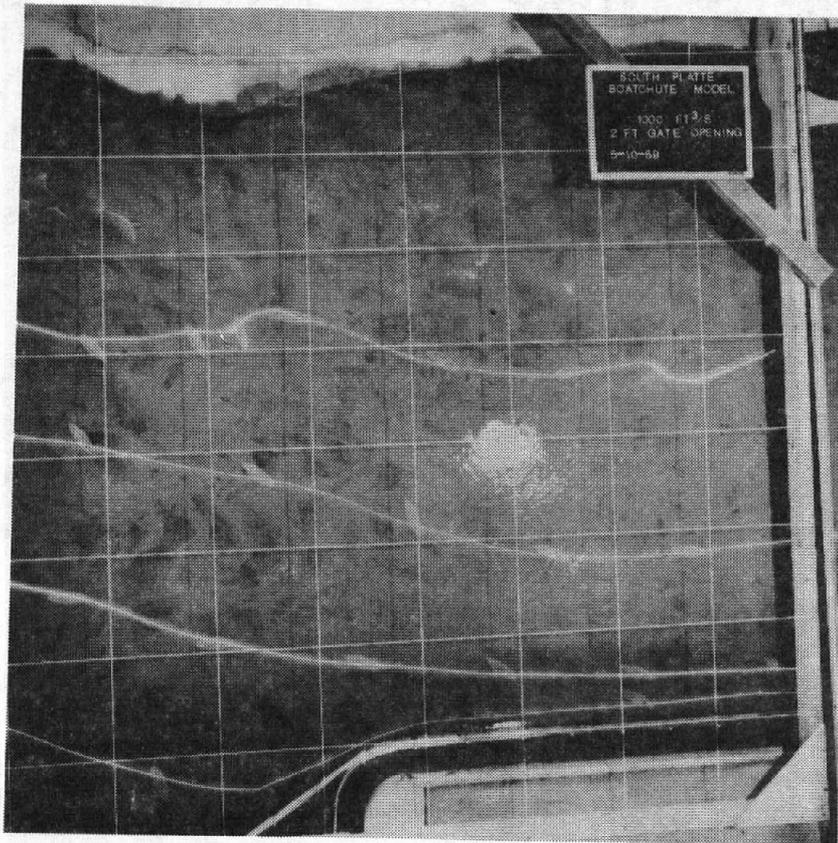


Figure 34. - Surface velocities in approach channel to Union Avenue dam, grid spacing = 18 feet,  $\Delta t = 8.5$  seconds,  $Q = 1,000 \text{ ft}^3/\text{s}$  (flow left to right).

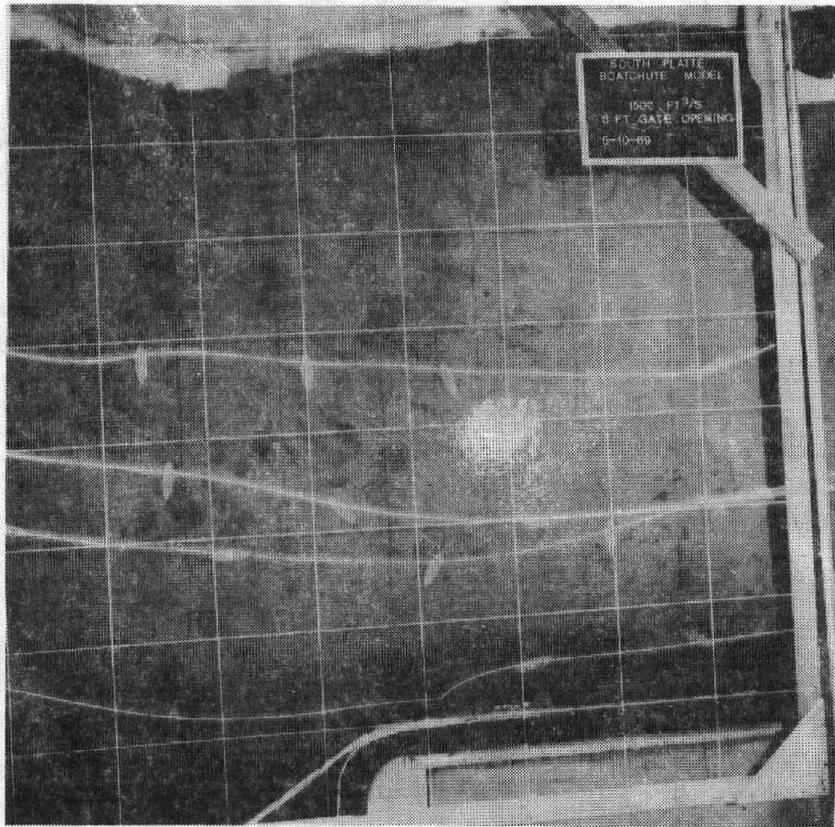


Figure 35. - Surface velocities in approach channel to Union Avenue dam, grid spacing = 18 feet,  $\Delta t = 8.5$  seconds,  $Q = 1,500 \text{ ft}^3/\text{s}$  (flow left to right).

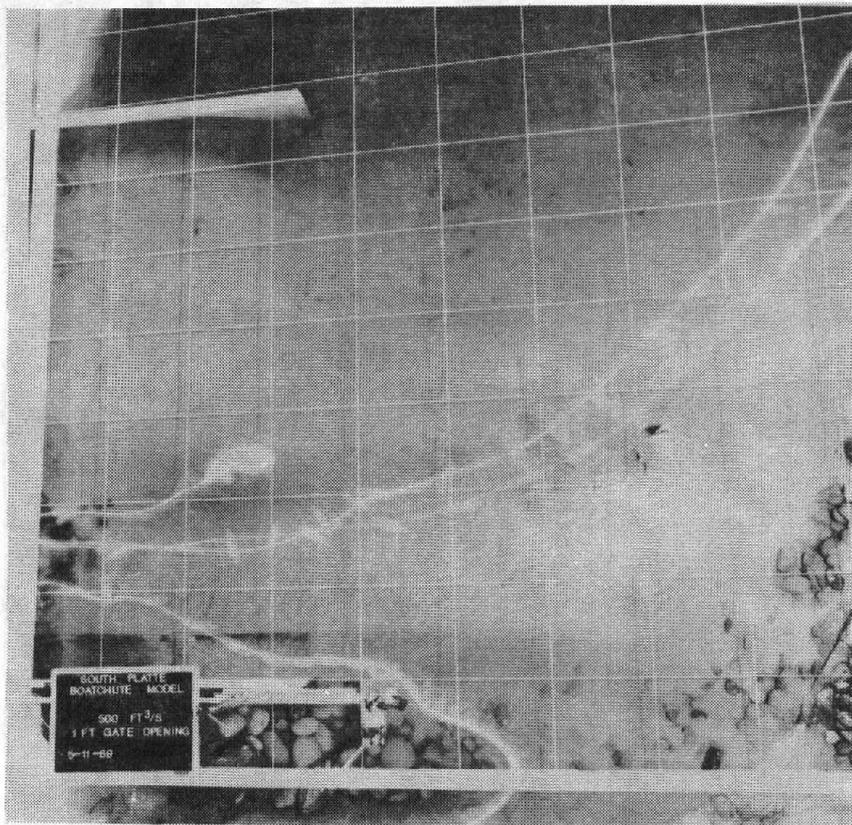


Figure 36. - Surface velocities downstream of boatchute 1, grid spacing = 18 feet,  $\Delta t = 3.4$  seconds,  $Q = 500 \text{ ft}^3/\text{s}$  (flow left to right).

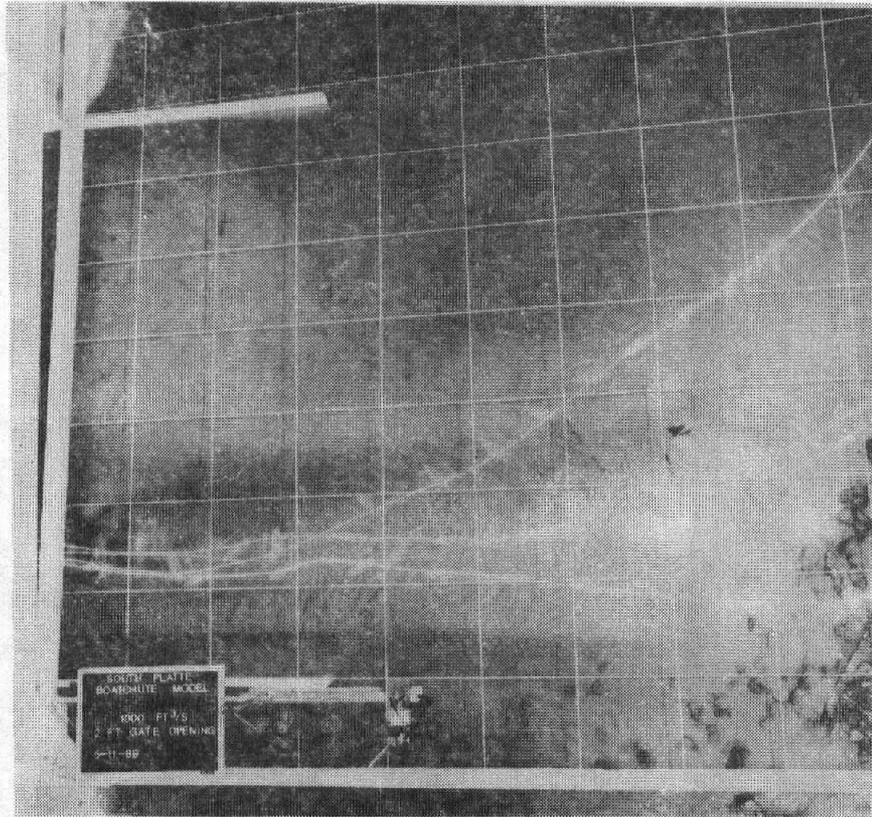


Figure 37. - Surface velocities in pool downstream of boatchute 1, grid spacing = 8 feet,  $\Delta t = 3.4$  seconds,  $Q = 1,000 \text{ ft}^3/\text{s}$  (flow left to right).

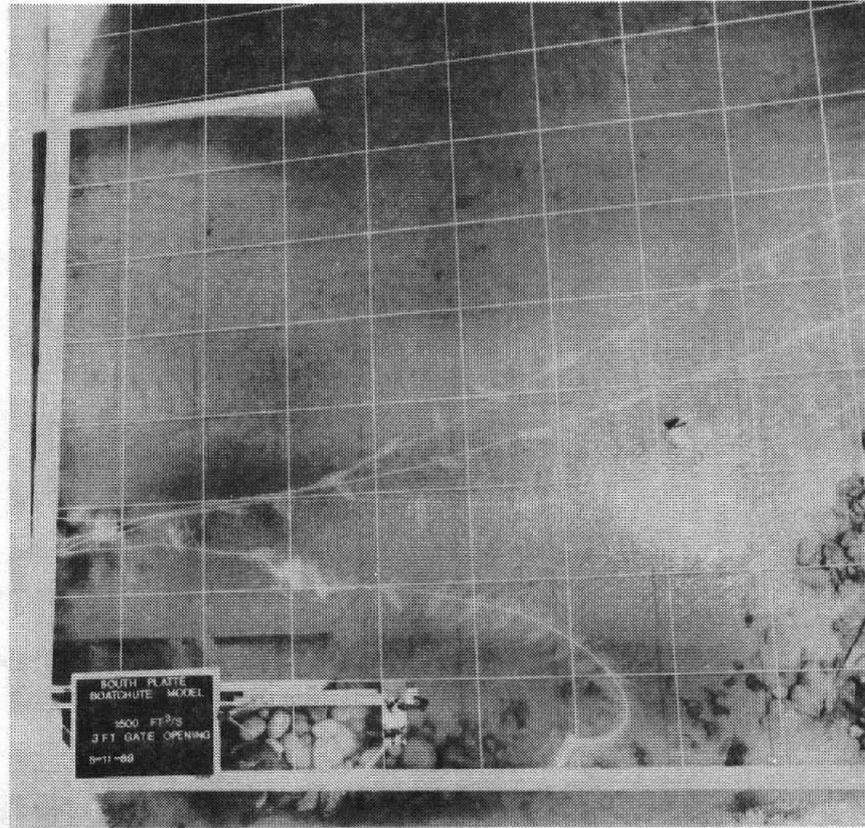


Figure 38. - Surface velocities downstream of boatchute 1, grid spacing = 18 feet,  $\Delta t = 3.4$  seconds, and  $Q = 1,500 \text{ ft}^3/\text{s}$  (flow left to right).

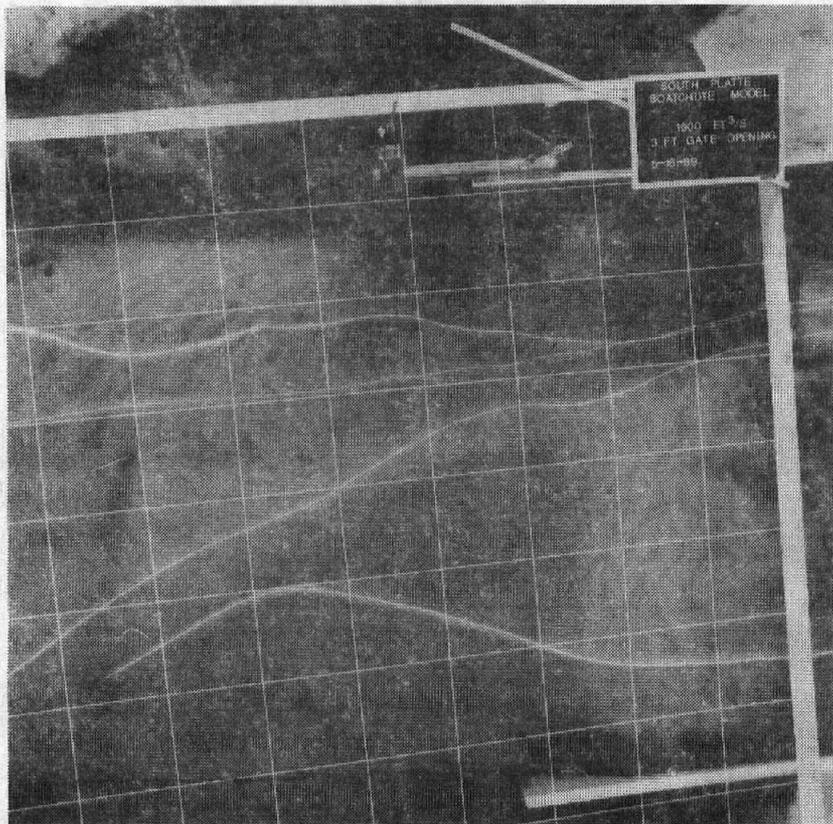


Figure 39. - Streak lines in first pool between boatchutes 1 and 2,  $Q = 1,500 \text{ ft}^3/\text{s}$  (flow right to left).

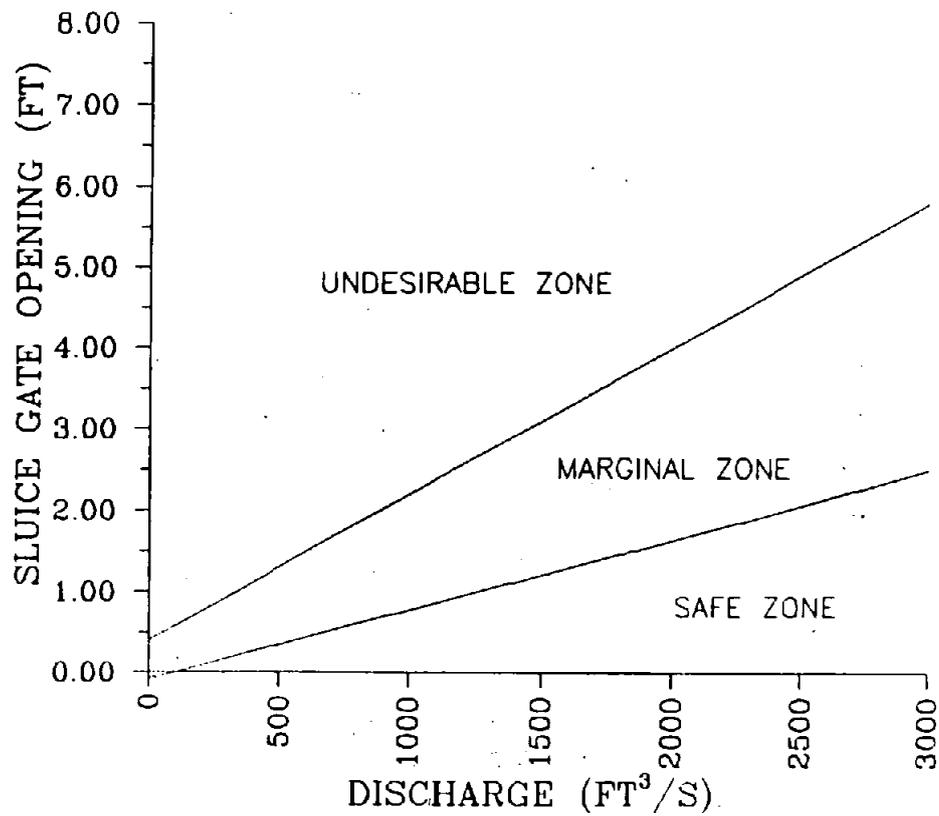


Figure 40. - Boating conditions with open-bar barrier in front of primary Englewood intake.  
 (This design was not selected.)

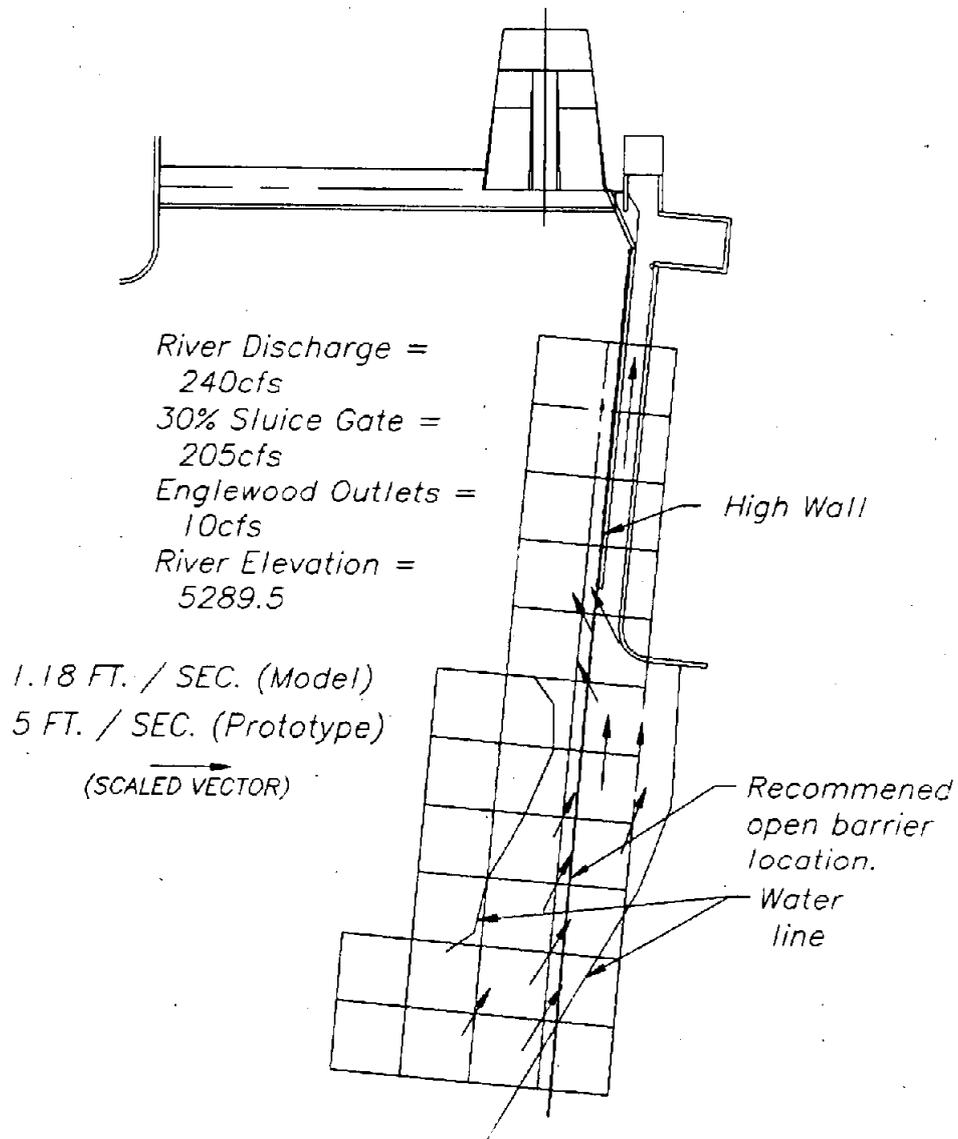


Figure 41. - Approach flow velocities upstream of Englewood intake ( $Q = 240 \text{ ft}^3/\text{s}$ ).

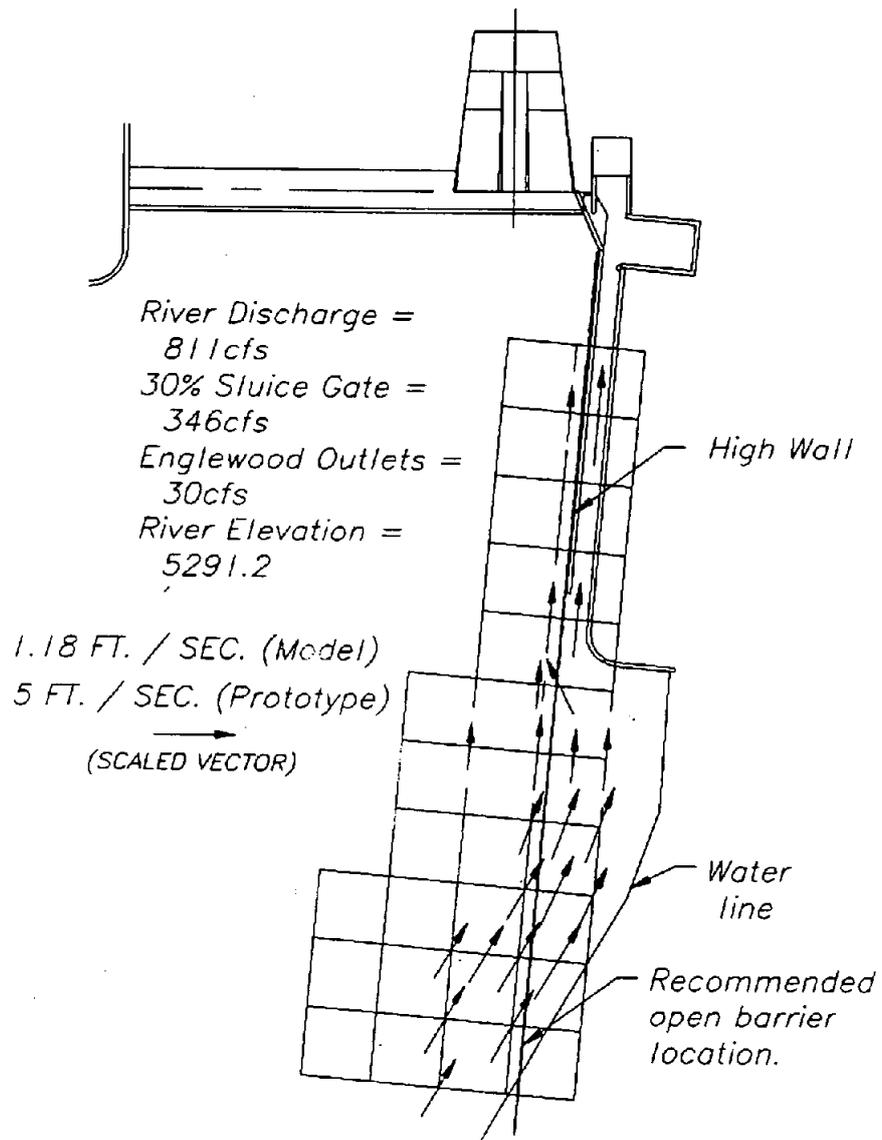


Figure 42. - Approach flow velocities upstream of Englewood intake ( $Q = 811 \text{ ft}^3/\text{s}$ ).

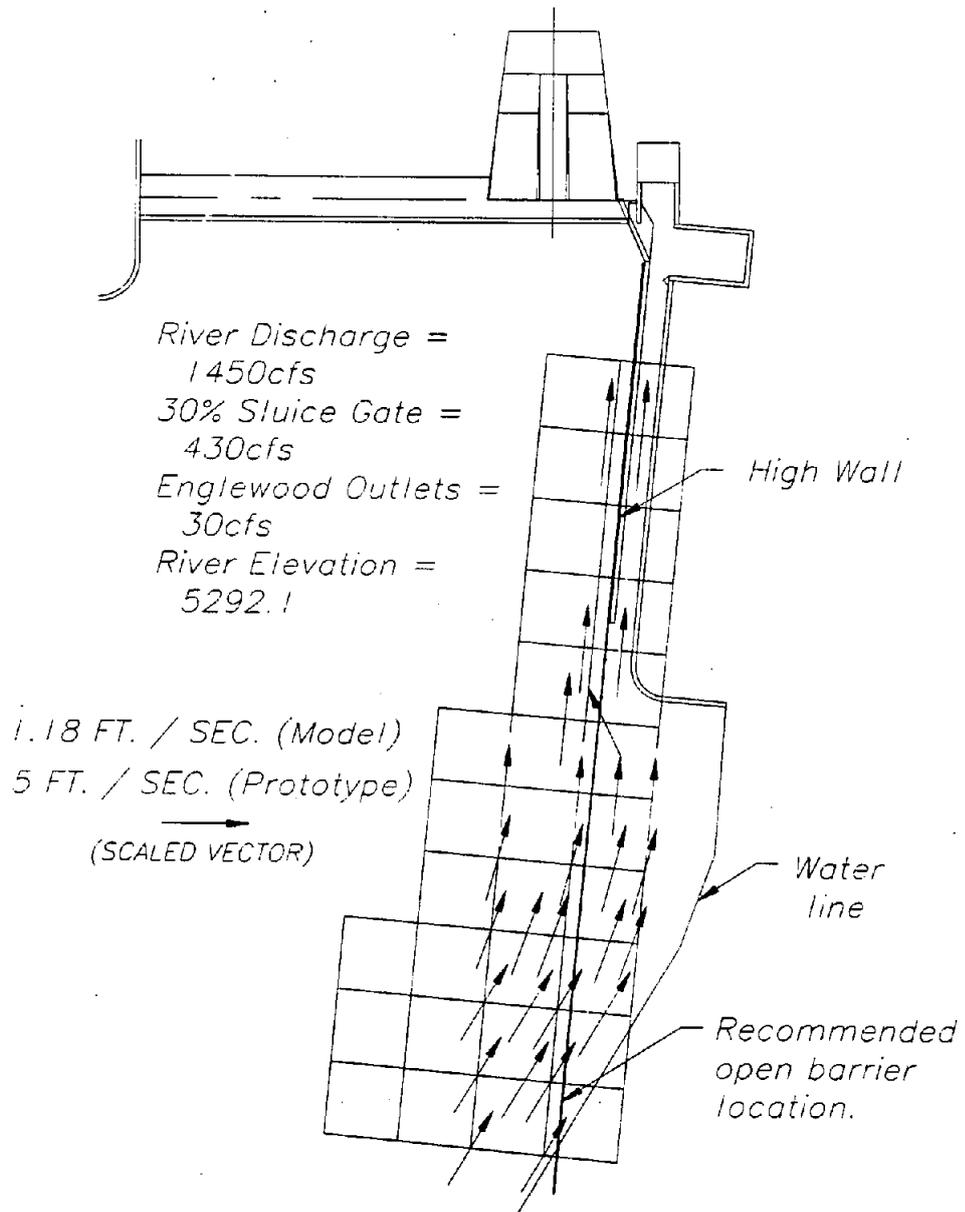


Figure 43. - Approach flow velocities upstream of Englewood intake ( $Q = 1,450 \text{ ft}^3/\text{s}$ ).

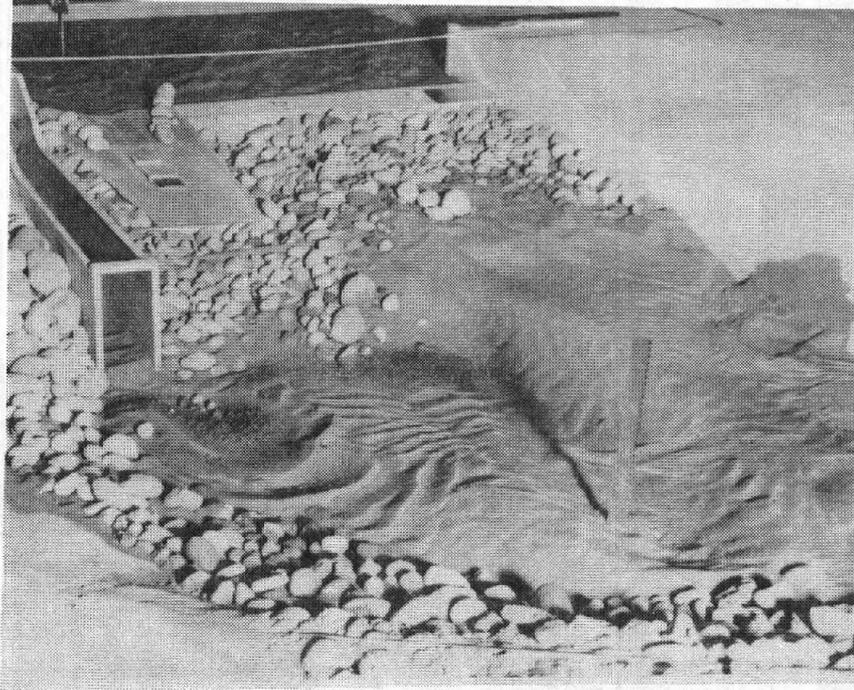


Figure 44. - Area between two boatchutes prior to sediment test.

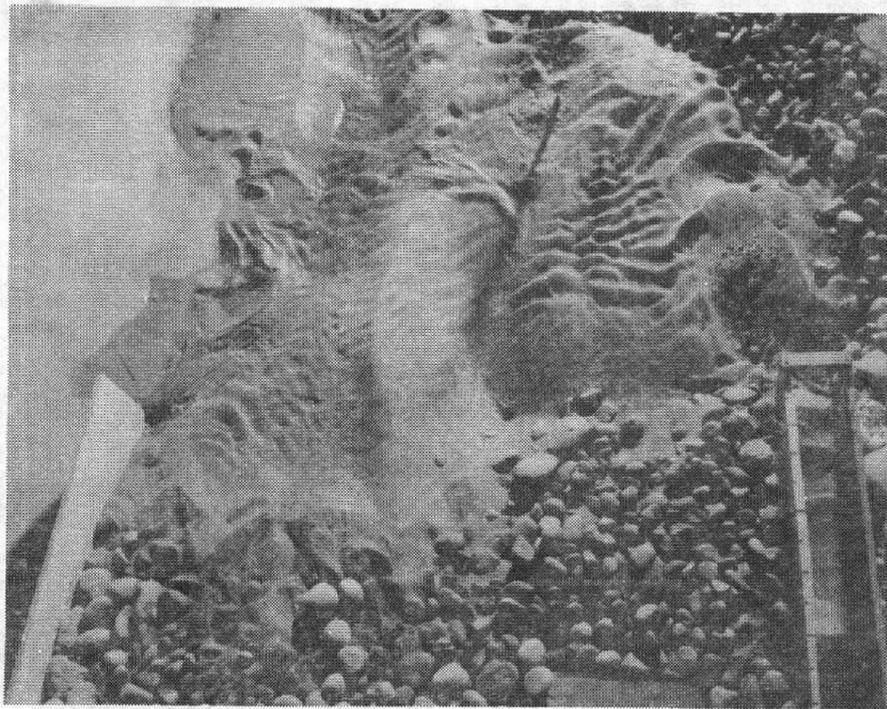


Figure 45. - Overhead view of area between boatchutes prior to sediment test.

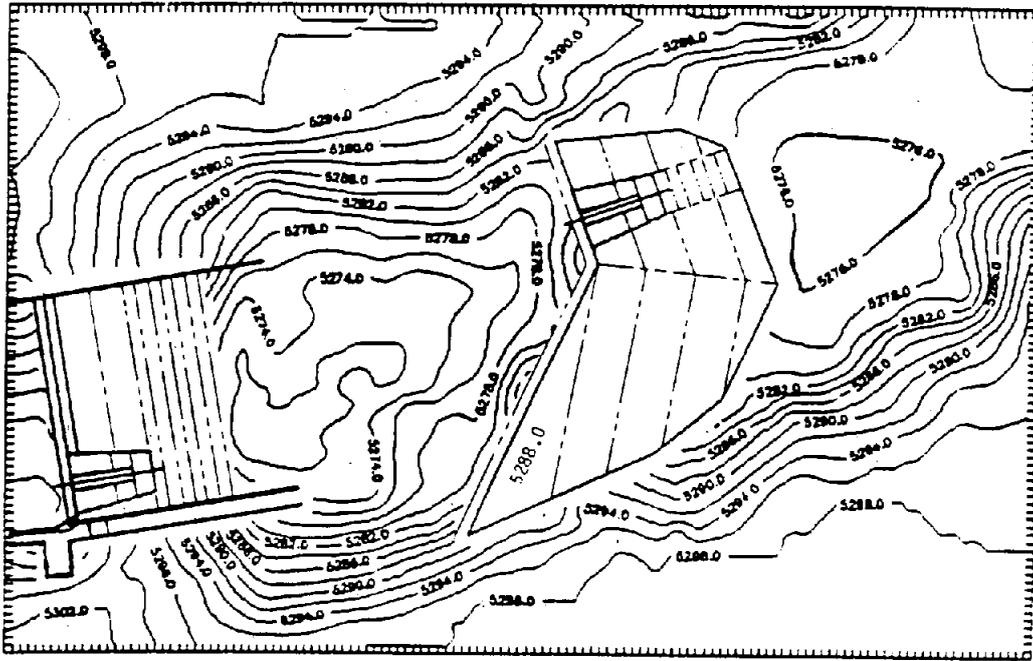


Figure 46. - Contours of pool area between boatchutes 1 and 2 prior to sediment test.  
(Contour interval is 2 feet.)

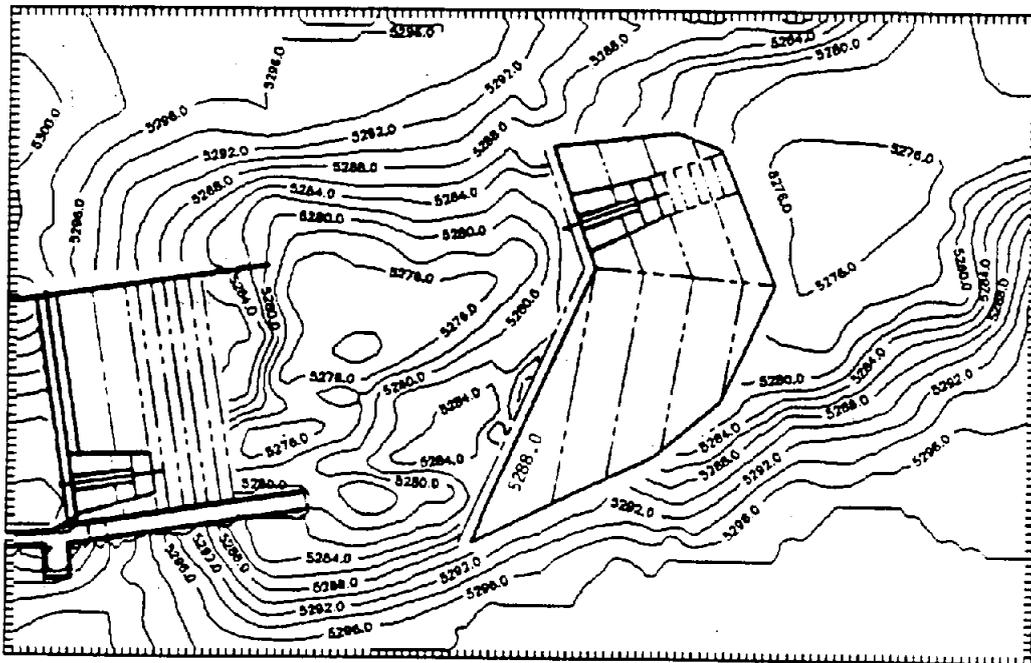


Figure 47. - Contours of pool area between boatchutes 1 and 2 after sediment test.  
(Contour interval is 2 feet.)

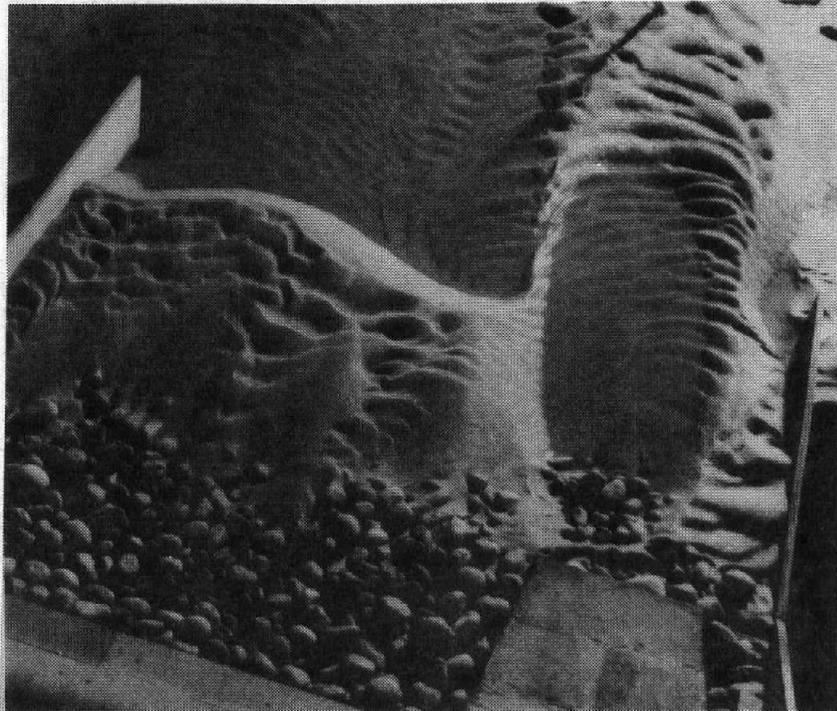


Figure 48. - Area between boatchutes after sediment testing.

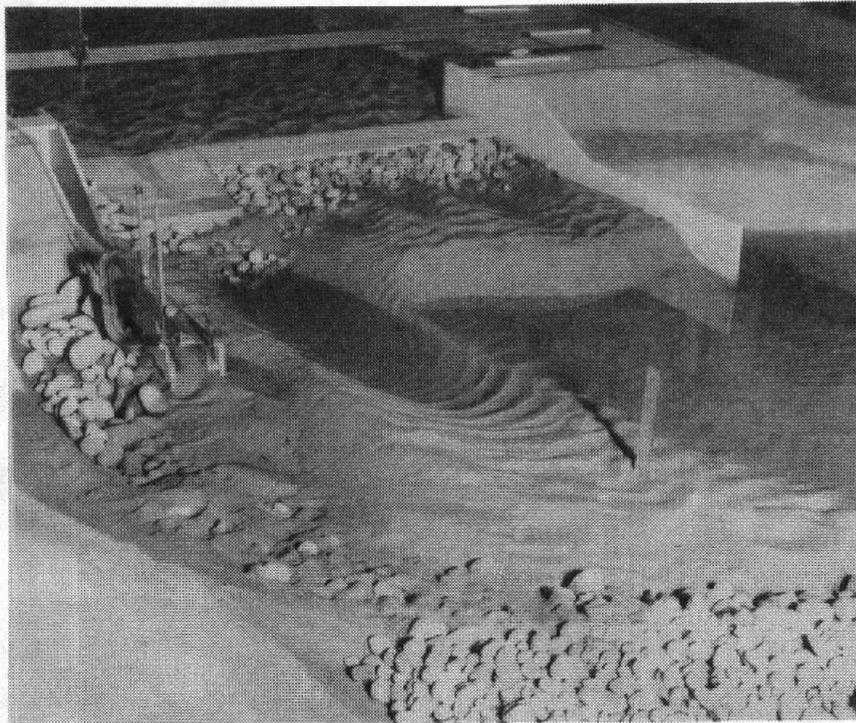


Figure 49. - Deposition after sediment test.

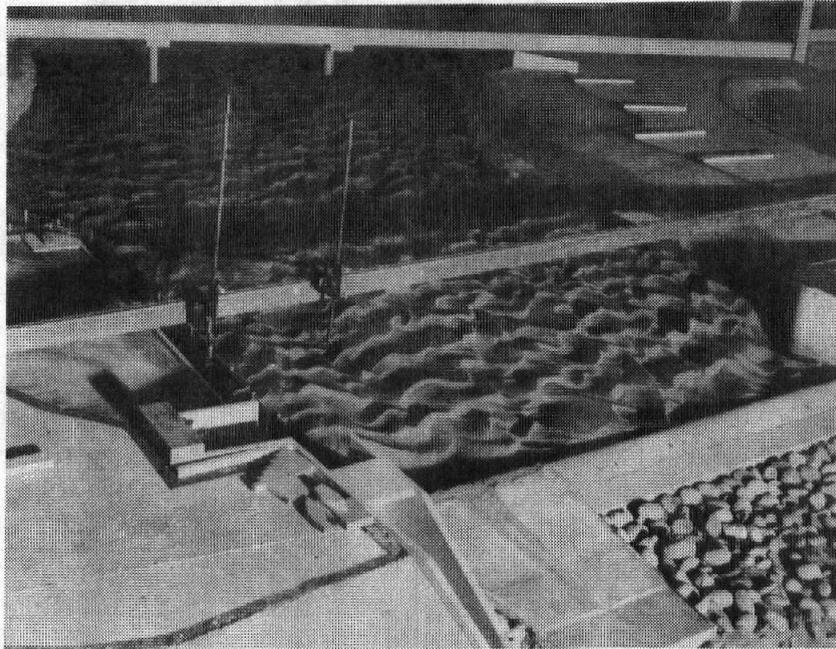


Figure 50. - Dunes formed upstream of boatchute 1 after sediment test.

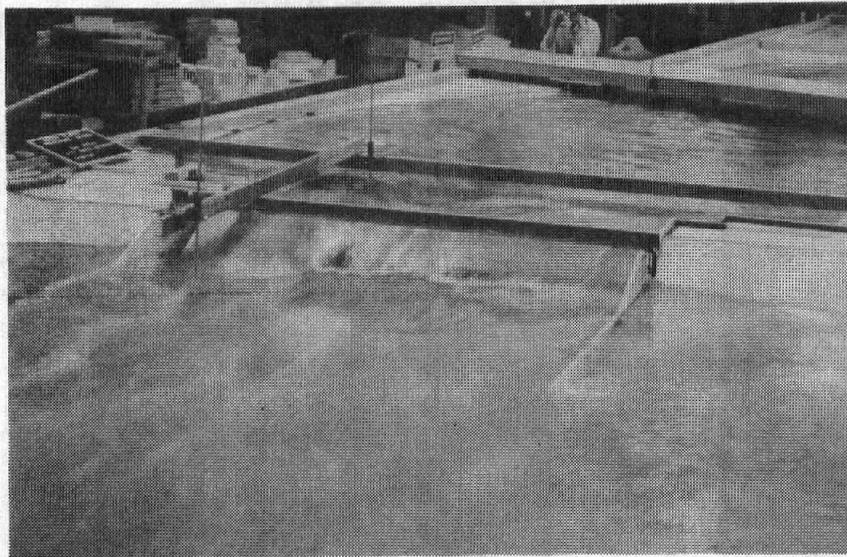


Figure 51. - Model with flow at  $16,400 \text{ ft}^3/\text{s}$  (COE 100-year flood) (final design).

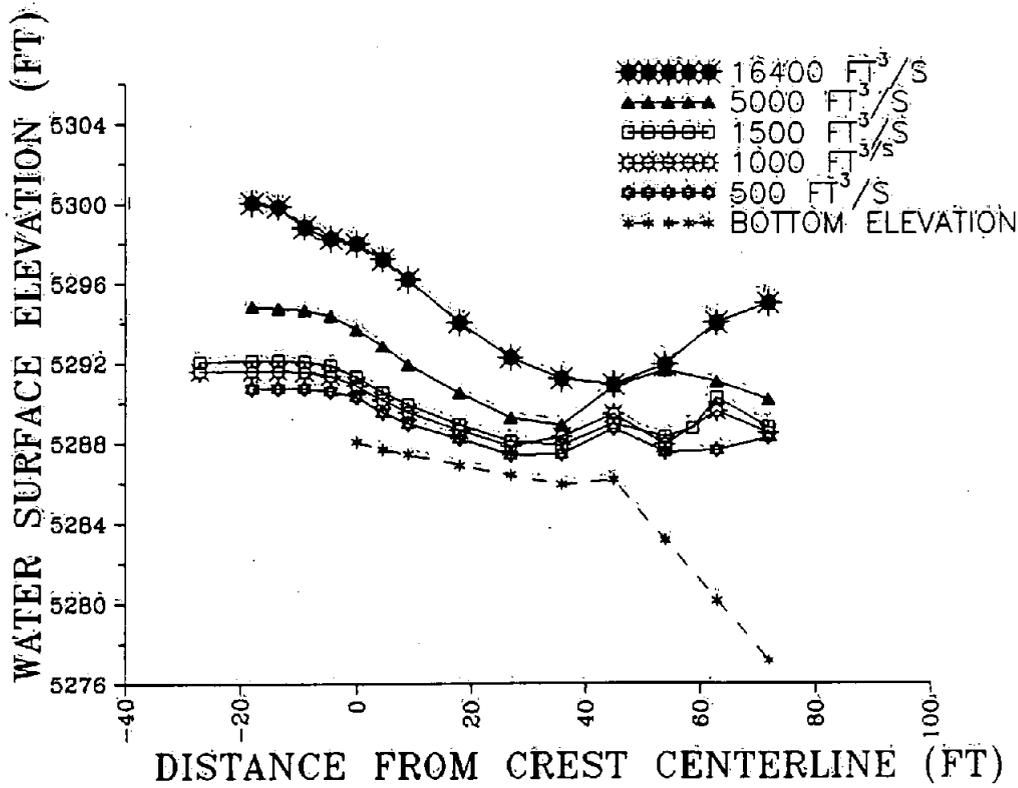


Figure 52. - Water surface profiles through boat chute 1.

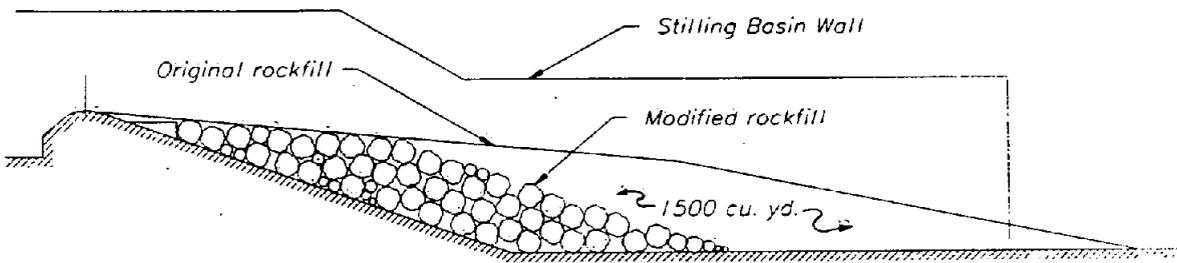


Figure 53. - Section through Union Avenue dam showing reduction in rockfill from original boat chute design.

### **Mission of the Bureau of Reclamation**

*The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.*

*The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.*

*Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.*

A free pamphlet is available from the Bureau entitled "Publications for Sale." It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-7923A, PO Box 25007, Denver Federal Center, Denver CO 80225-0007.