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GLEN CANYON DAM MULTI-LEVEL INTAKE STRUCTURE HYDRAULIC MODEL STUDY

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13. ABSTRACT ***(Maximum 200 words)***

A physical model was used to study the hydraulic characteristics of the proposed multi-level intake structure for Glen Canyon Dam. The multi-level intake structure is an uncontrolled overflow weir which permits selective withdrawal of warm surface water, when needed. The design includes the installation of an upper trashrack structure on top of the existing trashrack structure. In addition, a gate internal to the trashrack structure will be used to block the low-level intake during selective withdrawal operations. The gate will be raised to block the upper intake when low-level withdrawals resume. A 1:20 scale hydraulic model of a single penstock intake and trashrack structure was tested to determine the additional head losses associated with the intake modifications. Other features studied include vortex formation potential, internal and external velocity fields, submergence criteria, and water hammer pressures. All tests were conducted in an unstratified reservoir.

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by

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Denver, Colorado

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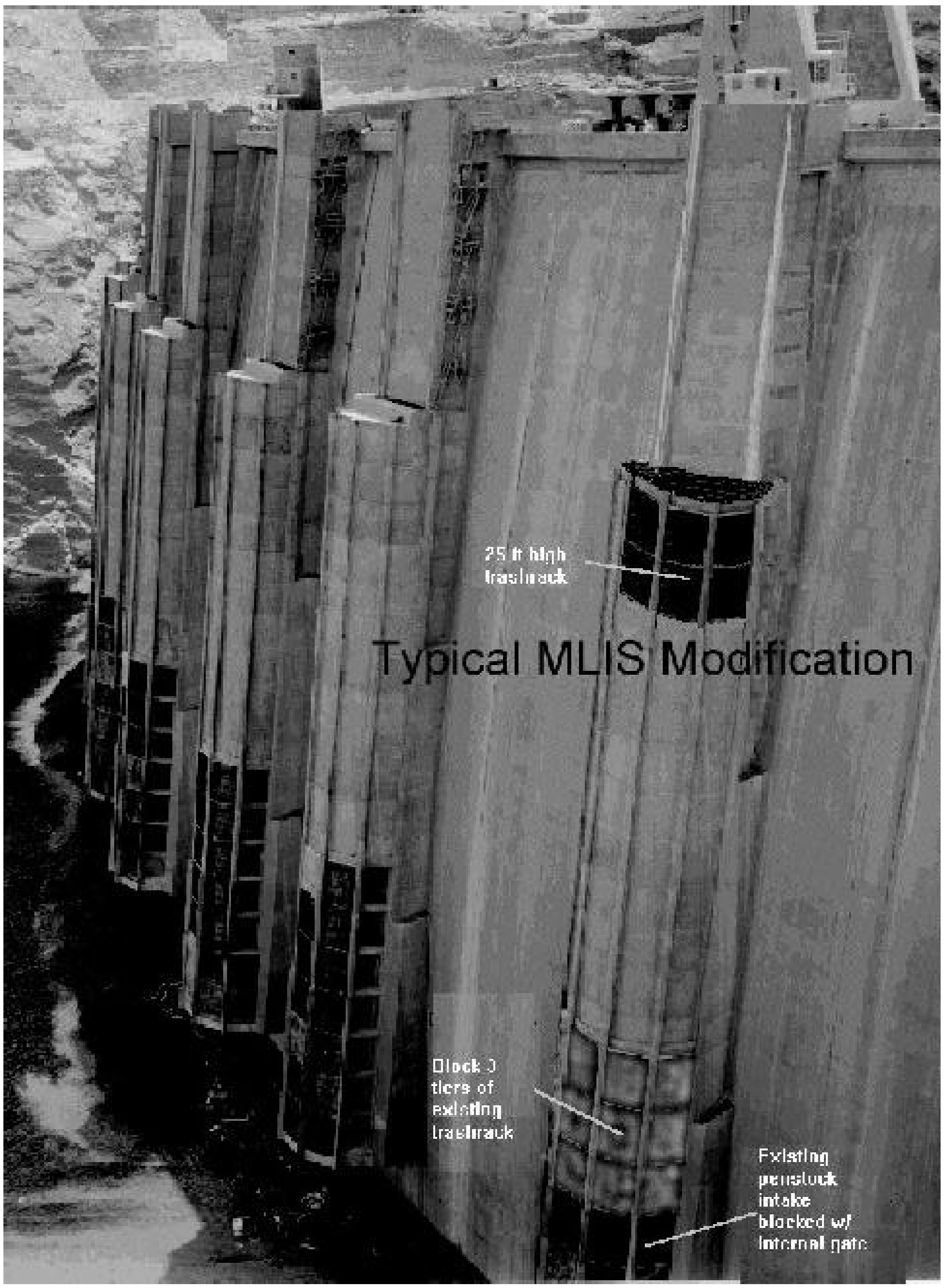
UNITED STATES DEPARTMENT OF THE INTERIOR ′ BUREAU OF RECLAMATION

U.S. Department of the Interior
Mission Statement

The Mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

ACKNOWLEDGMENTS

Wayne Delzer was the team leader for this project. Eliane Guiny, a summer intern, collected most of the data and wrote several sections of the report. Joe Kubitschek and Wayne Delzer peer reviewed the report, and Brent Mefford and Tony Wahl provided technical assistance for the study. Neal Armstrong was the craftsman responsible for model construction. Dean Connor and Thaxson Patterson were the hydraulic technicians responsible for the instrumentation used during this study. Robert Rood was the technical writer/editor and Tyna Petersen was the editorial assistant.



Frontispiece.—Digital image modified to illustrate the proposed Multi-Level Intake System for the eight penstock intakes at Glen Canyon Dam.

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PURPOSE

This hydraulic model study was conducted to collect hydraulic design data for the proposed Glen Canyon Dam Multi-Level Intake Structure (MLIS) and to develop modifications, if necessary, to improve hydraulic performance. The hydraulic data obtained from the model study were head losses, submergence criteria, near-field velocities, vortex formation potential, and qualitative water hammer pressures.

APPLICATION

Hydraulic model studies were conducted specifically for one penstock intake structure on Glen Canyon Dam. The results from this study will be used for the MLIS design for the Glen Canyon power penstock intakes. The differential head across the structure is an important component in the design of the modified trashrack structure and in developing operating criteria. MLIS designers were especially interested in how an increase in diffferential head might impact the structural integrity of the existing intake structure. The head loss results from the study will be useful for the design of similar uncontrolled selective withdrawal structures. However, because of uncertainties in modeling vortex formation, prototype observations are recommended to confirm that the 1:20 model scale was adequate for accurately predicting vortex formation and strength.

INTRODUCTION

Glen Canyon Dam is located on the Colorado River in north-central Arizona, 2 miles northwest of Page, Arizona (figure 1). Project construction was completed in 1964. The dam is a constant radius concrete arch with fillets. It has a structural height of 710 feet (ft), a crest elevation of 3715 ft, a crest length of 1,560 ft, a crest width of 25 ft, and a base width of 300 ft. Glen Canyon Dam is the fourth highest dam in the United States.

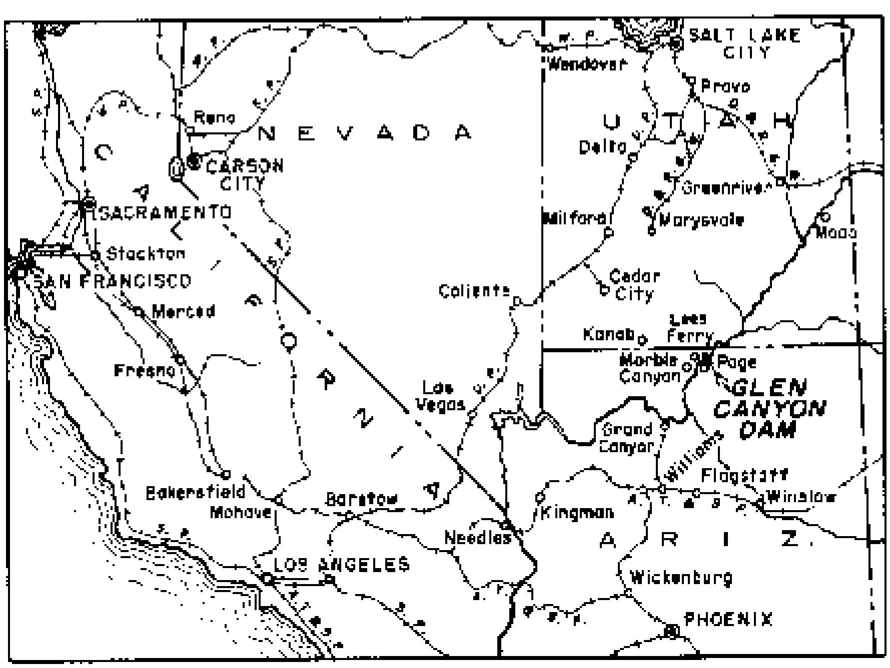

Figure 1.—Glen Canyon Dam location map.

The reservoir impounded by the dam is Lake Powell, the second largest in America, with a storage capacity of 27 million acre-ft at the maximum water surface elevation of 3700 ft.

Eight 15-ft-diameter penstocks deliver water to the 120 MW turbines. The centerline of each penstock intake is at elevation 3470. Elevation 3490 is the minimum water surface for power operations. All eight turbines in the powerhouse are rated at 155,000 HP at 450 ft of head. Each penstock intake is equipped with a fixed-wheel hydraulically operated gate which can operate under unbalanced head. The gates are located on the upstream face of the dam.

Each penstock intake is protected by a trashrack structure (figure 2). The top of the trashrack structure is at elevation 3652. The intake opening at the bottom of the trashrack structure is protected by trashracks that extend from elevation 3450 to about elevation 3537.5. Above the trashracks, concrete panels block the flow into the structure from elevation 3537.5 to the top of the trashrack structure. Guides and seats for stop logs were installed upstream from the fixed wheel gate to provide a means for inspecting and maintaining the gate frames and guides.

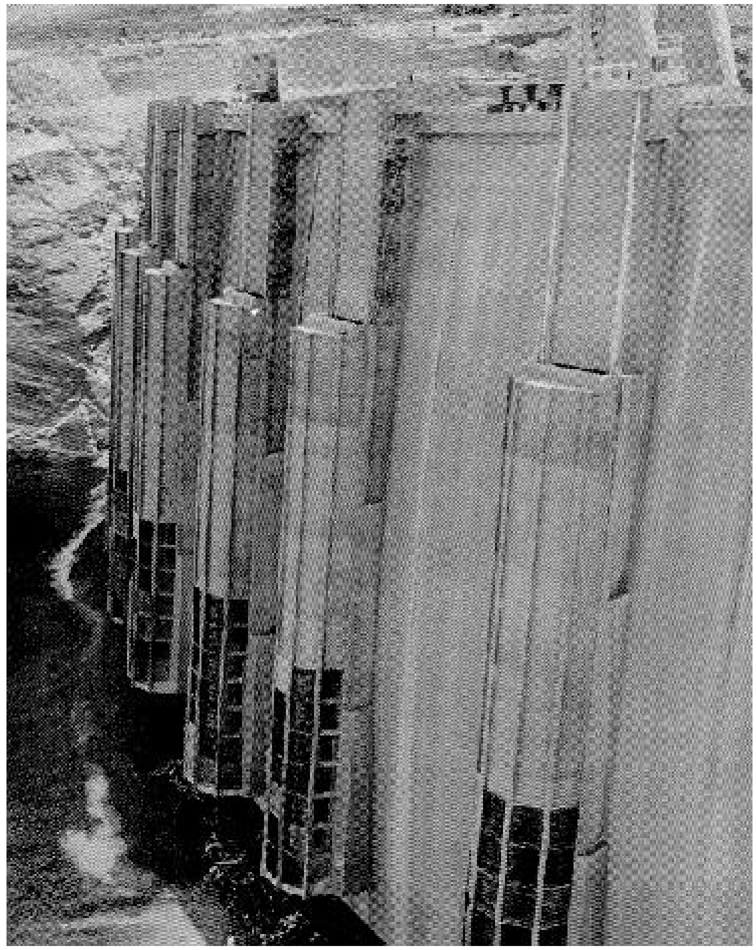

Figure 3.—Photograph of trashrack structures for the penstock intakes at Glen Canyon Dam.

PROJECT BACKGROUND

Since construction of the dam, the natural annual cycle of the temperature in the Colorado River below Glen Canyon Dam has been drastically modified. Pre-dam temperatures of the Colorado River would vary from freezing in the winter to about 85EF (29.4EC) in the summer. Now the water released downstream is a near-constant 45EF (7.2EC) because water is withdrawn through the eight low-level penstock intakes. As the water flows downstream, it gradually warms to about 60EF, which is not warm enough to allow endangered native fish to successfully reproduce in the mainstem of the Colorado River. This cold water is tolerated by the nonnative trout fishery, but is a threat for the native fish, namely, humpback chub (*Gila cypha*) and Colorado pike minnow (*Ptychocheilus lucius*). Research suggests that warmer releases would improve growth rates of the cold-water trout fishery immediately below the dam and enhance spawning and rearing habitat for endangered fish species farther downstream. The goal of this project is to release water at a temperature that benefits both the native and nonnative fishery, while keeping the unwanted nonnative warm water species from moving upstream from Lake Mead.

The U.S. Fish and Wildlife Service issued a Biological Opinion in 1994 recommending that the Bureau of Reclamation (Reclamation) investigate potential selective withdrawal modifications for Glen Canyon Dam. Selective withdrawal provides the capability for project operators to release water from several levels in the reservoir. The proposed penstock intake modifications will allow warm surface water to be released. Reclamation has investigated several structural modifications to provide various degrees of selective withdrawal. In the report, *Documentation of Feasibility Designs for Glen Canyon Multi-Level Intake Structure* (Reclamation 1997) prepared by Reclamation's Technical Service Center, three alternatives were developed to provide selective withdrawal to the existing penstock intakes.

An uncontrolled overdraw alternative (figure 3) was selected for implementation by Reclamation's Upper Colorado Regional Office. As part of the design process, the Water Resources Research Laboratory was tasked with performing a hydraulic model study of the proposed modifications to a single penstock intake and trashrack structure.

DESCRIPTION OF THE PROPOSED DESIGN

A detailed description of the uncontrolled overdraw proposal is given in the report entitled *Documentation of Feasibility Designs for Glen Canyon Multi-Level Intake Structure* (Reclamation 1997).
This alternative withdraws water through an opening cut in the top of the existing trashrack structure; the top cover and the top 21 ft of the existing trashrack structure would be cut and removed providing a semicircular intake. The cross sectional area of the opening is about 360 square feet (ft^2), including the open area of the fixed wheel gate slot. The inside radius of the opening will be about 14.5 ft. A new trashrack with a removable lid will be installed to permit passage of a low-level pressure control gate. The 11.5-ft-radius pressure control gate will also block the upper intake during cold water withdrawals (figure 3).

The existing trashrack structure and the new trashracks provided at the top of the structure are designed for a 20-ft positive differential head. A control gate will be designed for a maximum differential head of 10 ft. The control gate will contain relief panels with shear pins designed to fail at approximately 5 ft of head differential. The relief panels are necessary to protect the structure from excessive head differential caused by misoperation of the fixed wheel gate or powerplant.

CONCLUSIONS

For warm water withdrawal mode, the head losses through the MLIS are significant and will result in some loss of power revenues. At the design discharge of 4,000 feet^3/second (ft^3/sec), the additional head loss was about 3.7 ± 0.2 ft. The system head losses were minimally affected by different levels of submergence.

For warm water withdrawal mode, the head loss across the control gate was measured to assist with the design of the relief panel shear pins. At the design discharge of 4,000 ft^3/sec and 40 ft of submergence, the piezometric head drop across the control gate was 2.4 ± 0.1 ft. When the internal velocity head is included, the total head loss across the control gate is 3.8 ft. Thetotal head loss should be used in the design of the relief panel shear pins.

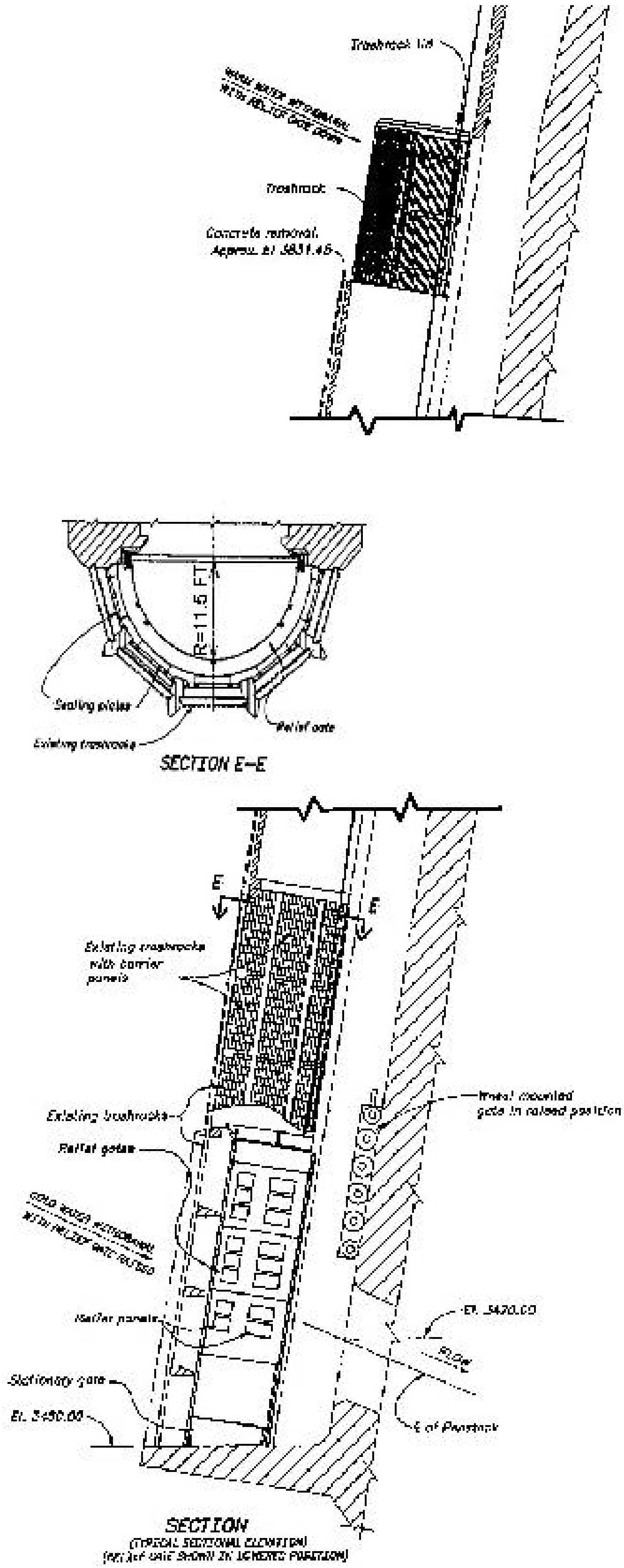

Figure 4.—Section views of the proposed uncontrolled overdraw MLIS for Glen Canyon Dam (not to scale).

Tests showed that for a 1-ft partial gate opening, the additional head loss was reduced by about 1.2 ft, or 30 percent for the design discharge. It is recommended that a *minimum* 1-ft gate opening be maintained to prevent vibration and hydraulic downpull on the control gate. Vibration and hydraulic downpull were not evaluated during model testing. Consequently, field evaluation of vibration and downpull is recommended.

For cold water withdrawal mode, the additional head loss through the MLIS was about 0.5 ± 0.1 ft at the design discharge of 4,000 ft^3/sec. This additional head loss is attributed to blocking 52 percent of the lower trashrack panels and adding the stationary gate section to the bottom of the trashrack structure.

Based on model observations, 40 ft is the recomended minimum submergence for the MLIS design discharge of 4,000 ft^3/sec. Operating at or above this submergence should prevent vortex formation.

A solid trashrack lid is recommended to force vortex formation outside the perimeter of the trashrack. This feature keeps vortices formation away from mechanical equipment. The solid lid resulted in an additional head loss of about 0.3 ± 0.2 ft. This additional head loss was reduced by increasing the trashrack height by 5 ft.

If the MLIS is operated at submergence levels less than 30 ft, air core vortices may occur.

Because of uncertainties in modeling vortices at a reduced scale, the minimum acceptable submergence should be reevaluated based on prototype observations of vortex formation.

A comparison of model and prototype head losses for the existing intake agreed reasonably well, considering model limitations, such as differences in Reynolds number, penstock diameter and length, roughness coefficients, and intake shape.

Velocity profiles collected inside and outside the trashrack structure were used to describe the head loss characteristics of the MLIS.

SIMILITUDE AND MODEL SCALE

For this model study, similitude was based on Froude law scaling, which is commonly used where inertial and gravitational forces control the flow. The model dimensions (geometric similitude) and hydraulic data (kinematic and dynamic similitude) can be scaled to prototype values using the relationships in table 1.

Table 1.—Geometric, Kinematic, and Dynamic Scaling Relationships

Dimensions	Ratio	Scaling Relationships (model : prototype)
Length (geometric)	$L_r = L$	1 : 20
Area (geometric)	$Lr = L_r{}^2$	1 : 400
Time (kinematic)	$T_r = L_r{}^{1/2}$	1 : 4.47
Velocity (kinematic)	$V_r = L_r{}^{1/2}$	1 : 4.47
Discharge (kinematic)	$Q_r = L_r{}^{5/2}$	1 : 1788.8
Pressure (dynamic)	$P_r = L_r$	1 : 20

A model scale of 1:20 was chosen so that the Reynolds numbers (ratio of inertial to viscous forces) were large enough to allow for accurate modeling of pipe friction losses, vortex formation, and water hammer pressures. A series of tests were conducted to assure that differences in model and prototype Reynolds numbers did not affect system head loss measurements.

MODEL SETUP

The model was designed to include a single penstock intake, trashrack structure, and about 220 ft of penstock. An intake transition used for a similar model study for Hungry Horse Dam (Kubitschek 1994) was reused for this study. This intake was similar to the Glen Canyon intake and was used to keep model construction costs to a minimum. Eight-inch, schedule 40 PVC pipe was used for the penstock piping. The inside diameter of the pipe scales to 13.2 ft, which is smaller than the 15-ft inside diameter of the Glen Canyon's penstocks. Again, the use of commercially available pipe kept construction costs down. These two deviations between model and prototype dimensions had little or no impact on this study because we were measuring the additional head loss associated with modifications to the intake structure. Since no changes in the intake or penstock configuration were made, their impact on the additional head loss measurements were negligible. However, these differences in pipe material and dimensions did affect the water hammer testing.

Discharge Measurements

The model discharge was controlled by means of a gate valve (figure 4) and was measured with a McCrometer propeller flowmeter with a 4-20 mA analog transmitter (figure 5). The propeller meter's analog output was connected to an integrating voltmeter which computed an average voltage output for 1,000 samples (figure 6). The typical standard deviation of the mean (SDOM) for the maximum discharge was ± 0.5 ft^3/sec (prototype). Throughout the model testing, average discharge and differential pressure measurements were collected simultaneously.

A series of calibration tests were conducted to develop a relationship between the model discharge and the flowmeter's voltage output:

$$y = 1250\,x - 623.8 \quad (R^2 = 1.00)$$

where: x = flowmeter output in volts
y = model discharge in gal/min

Figure 4.—Flow control valve.

Figure 5.—Propeller flowmeter with analog output.

The flow in gallons per minute was determined by measuring the total volume of water flowing through the meter for a period of 10 minutes. Likewise, a strap-on acoustic flowmeter was used as an independent check of the propeller flowmeter. A comparison of the propeller and acoustic flowmeters showed they agreed to within ± 2.3 percent over the range of flows used for this model study. The agreement between the two flowmeters was considered to be adequate to confirm the accuracy of the propeller flowmeter's volumetric totalizer.

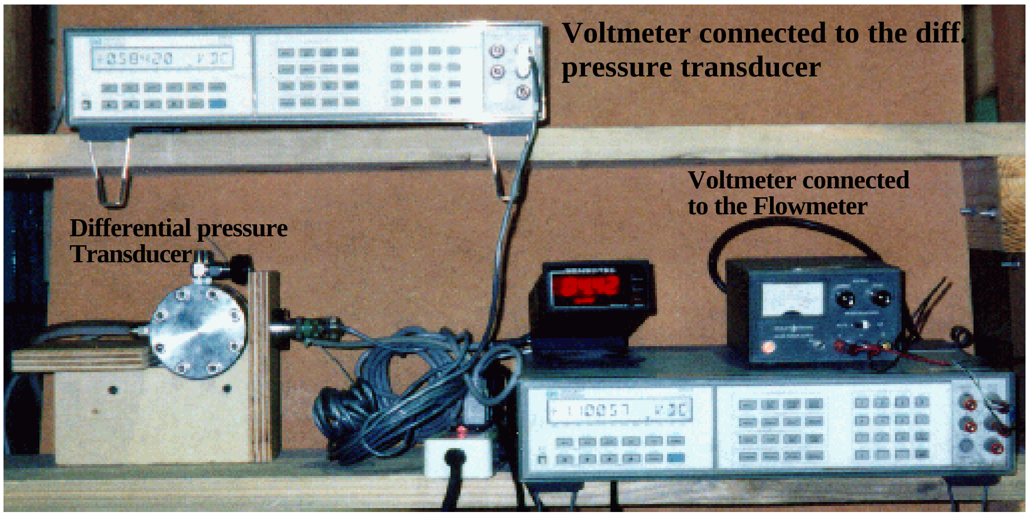

Figure 6.—Instrumentation setup for measurement of average differential pressure and discharge.

Measurement and Control of Water Surface Elevation

The reservoir water surface elevation was set using a hook gage in a stilling well. A variable-speed pump control system was used to maintain a constant water surface level during a test. The pump controller varied the pump speed as a function of the water level measured by a Druck pressure transducer (S/N 879273). The 4-20 mA output from the pressure transducer was used as input to an Omega CN76000 PID (Proportional Integral Derivative algorithm) controller. The output from the controller was used to adjust the pump speed to maintain a constant water surface level.

The PID control constants were determined using the Zeigler-Nichols tuning method (Stephanopoulos 1984). The pressure transducer used to sense model water surface level was connected to a data acquisition system. The tuning started with the proportional constant (K_p) was set equal to 15 and was reduced to a value where the pressure output began to oscillate. The K_p setting where oscillation began was 5. The ultimate gain (P_U) is the inverse of the K_p value, which was 0.20. The ultimate period (T_U) between oscillations was 32 seconds. The optimum control occurred using a no overshoot criterion. The PID constants were calculated using the P_U and T_U values. The proportional band constant (Pb1), integral constant (Res), and derivative constant (Rte) were calculated using the following relationships:

$$Pb1 = \frac{1}{0.2 \times \frac{1}{P_u}}, \quad Res = 0.33 \times T_u, \quad Rte = 0.5 \times T_u \tag{1}$$

The PID controller was set with Pb1 equal to 25.00, Res equal to 0.2 minutes, and Rte equal to 0.27 minutes. For these control constants, the model took about 5 minutes to reach a stable water surface when the model discharge or water surface level was adjusted. This control system was very effective, which resulted in very repeatable head loss measurements.

Differential Pressure Measurement

The pressure head drop between the reservoir and a cross section in the penstock was used to calculate the total system head loss. For this model, two methods were used to measure differential piezometric pressure head. A Sensotec 2 lb/in^2 differential pressure (DP) transducer was used along with a U-tube manometer. A series of calibration tests were conducted to develop a calibration equation for the DP transducer (S/N 299125). In general, the U-tube manometer was used as a check on the DP pressure transducer measurements.

Both the transducer and U-tube manometer were connected to a 4-port piezometer ring attached to the penstock and a piezometer connected to the reservoir. The penstock piezometer ring was located about 138 ft downstream from the penstock intake. The total system head loss was measured using the DP transducer. The pressure transducer's analog output was connected to an integrating voltmeter, which was used to measure an average and standard deviation of 1,000 readings of the voltage output (figure 6). During the calibration, a linear relationship between the transducer output, in volts, and the DP head, in inches, was determined to be:

$y = 11.125\,x \quad (R^2=1.0)$ where: y = DP head, inches of water
x = DP transducer output, volts

MODEL TESTS

Theory

Bernoulli's equation was applied to determine the system head losses for flow through the trashrack structure, penstock intake, and a length of penstock. Bernoulli's equation (equation 2) is of the form:

$$z_1 + \frac{P_1}{\gamma} + \frac{v_1^2}{2g} = z_2 + \frac{P_2}{\gamma} + \frac{v_2^2}{2g} + h_L \qquad \textbf{(2)}$$

where: z_1 = reservoir elevation, ft
z_2 = penstock piezometer ring elevation, ft
P_1/γ = pressure head at reservoir piezometer location, ft
P_2/γ = pressure head at piezometer ring location, ft
$v_1^2/2g$ = velocity head at reservoir piezometer location, ft
$v_2^2/2g$ = velocity head at piezometer ring location, ft
h_L = total head loss, ft

Solving for the head loss, h_L, assuming v_1 is zero, results in equation 3:

$$h_L = \left(\frac{P_1}{\gamma} + z_1\right) - \left(\frac{P_2}{\gamma} + z_2\right) - \frac{v_2^2}{2g} \qquad or \qquad h_L = \Delta PH - \frac{v_2^2}{2g} \tag{3}$$

where: ΔPH is the differential piezometric head

ΔPH was measured using a U- tube manometer and DP transducer, and the velocity head was calculated from the known penstock diameter and discharge. A system head loss coefficient, K_e, was computed as the ratio of head loss measured at the piezometer ring divided by the velocity head in the penstock (equation 4):

$$K_e = \frac{h_L}{\frac{V_p^2}{2g}} \tag{4}$$

The DP measurements were a direct measure of ΔPH. The velocity head (h_v) at the piezometer ring is defined in equation 5:

$$h_v = \frac{V_p^2}{2g} \quad or \quad h_v = \frac{Q^2}{2gA_p^2} \tag{5}$$

where: Q is the discharge, ft^3/sec
V_p is the velocity in the penstock, ft/sec
A_p is the internal cross sectional area of the penstock, ft^2

Additional head loss coefficients for each intake modification were calculated using the following relationship (equation 6):

$$K_e^{\Delta} = K_{e_{\mathrm{mod}_i}} - K_{e_{baseline}} \tag{6}$$

Head Loss Versus Discharge Relationships

The first model tests were conducted for the existing intake structure configuration. The system head loss was measured between the reservoir and a piezometer ring on the penstock, which was located about 138 ft (prototype) downstream from the dam face. These tests were conducted to determine the baseline head losses which were used to determine the additional head loss associated with the proposed intake modifications. The additional head loss was calculated by subtracting the baseline head loss from the measured head loss for each intake modification tested. Head losses were determined for a range of discharges up to the design capacity of the penstock, 4,000 ft^3/sec.

The following modifications were tested in this model study:

Modification Zero:	Existing intake configuration (figure 7).
Modificaton One:	Testing was not completed because modification two was made.
Modification Two:	Control gate in raised position blocking the upper trashrack. Solid panels blocked a portion of the lower trash racks. The fixed portion of the control gate was installed on the base of the trashrack structure.
Modification Three:	Surface withdrawal through a 20-ft-high trashrack. Control gate was in the lowered position.
Modification Four:	Same as modification three, except with the addition of a solid trashrack cover to mitigate vortices.
Modification Five:	Surface withdrawal through a 25-ft-high trashrack with a solid cover. The upper level of the trashrack was blocked, fixed-gate in place, and control gate in down position (figure 8).
Modification Six:	Same as modification five, except with a 1-ft opening in the lowered control gate (figure 9).
Modification Seven:	Determine total head loss across reservoir and control gate. This involved adding a piezometer tap on the floor of the intake structure 3.3 ft (prototype) downstream from the stationary gate, on the structure's centerline.

Evaluating Vortex Formation

The study of vortex formation potential was conducted at the same time as the collection of the head loss data versus discharges. Flow visualization techniques were used to determine vortex strength using a classification system developed at the Alden Research Laboratory (ARL) by Hecker (1981), see figure 10. Video tape and photographs were used to document vortex strength and location.

Scale modeling of vortices is difficult because there is no single similitude law that accurately correlates observed model vortices to those observed in the prototype. The problem is that a Froude-based model does not accurately reproduce hydraulic phenomenon, such as vortices, which are affected by Reynolds and Weber numbers. Consequently, the vortex evaluation from the model study is qualitative. However, based on 16 cases where model-prototype data were available, Hecker (1981) reports:

> In general, final intake designs which were developed from Froude scale model tests to be vortex free were indeed vortex free in the prototype, and those that had weak vortices in the model had weak prototype vortices. In particular, there is no reported case in which a negligible model vortex corresponded to a sufficiently strong prototype vortex that it produced operating problems.

Hecker suggests that a dye core vortex (type 3) be used as the design limit for hydraulic modeling when air core (type 5) would negatively impact operations.

Prototype observations of vortices at Glen Canyon Dam were reported by project personnel under conditions of near minimum submergence (~23 ft). The vortex core was reported to be 1-ft in diameter with a 6-ft core depth (Hecker 1981).

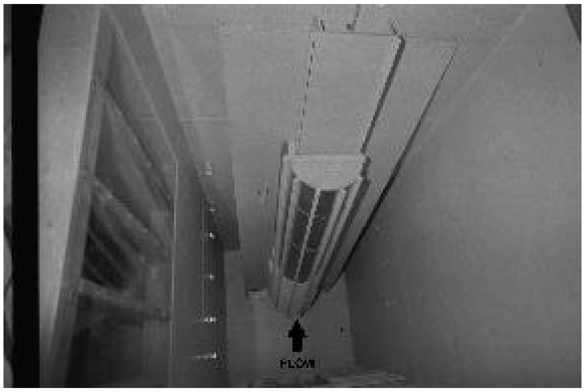

Figure 8.—Photograph of existing intake configuration (modification one).

Figure 9.—Photograph of 25-ft-high trashrack (modification five).

Figure 10.—Photograph of 1-ft gate opening with trashracks removed (modification six).

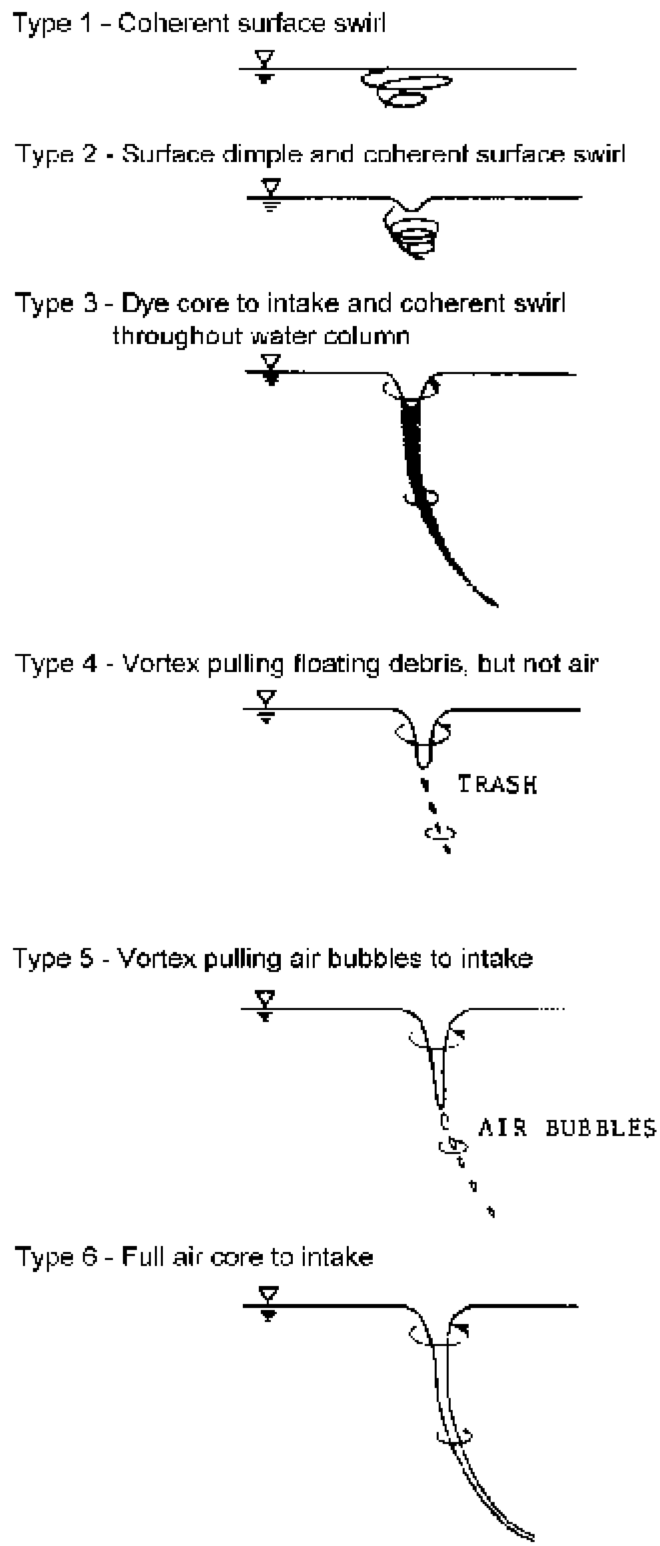

Figure 11.—ARL vortex type classification (Hecker 1981).

Velocity Profile Measurements

Three-dimensional velocity profiles were collected for the existing conditions and for the modifications under steady state conditions using a Sontek acoustic Doppler velocimeter (ADV). The ADV uses remote sensing techniques to simultaneously measure three-dimensional velocity components (x, y, and z) using a single sampling volume. Velocity profiles were collected for the following conditions:

Existing conditions - Velocity profiles were collected in front of the trashrack and the baffle to evaluate model performance.

Selective withdrawal structure with control gate in lowered and raised position - External and internal velocity profiles were collected at several different discharges and water surface elevations. External velocity profiles were measured along the centerline of bay 3 about 6 ft from the trashrack structure. Internal velocity profiles were done along the trashrack centerline inside the structure at a distance of 8 ft from the upstream face of the fixed wheel gate.

Figure 11 illustrates the velocity profile coordinate system for the Sontek ADV. The orientation of the three velocity components is defined as positive with Vx in the upstream direction, positive Vy is toward the right looking downstream, and positive Vz is upward.

Velocities collected for this study are a good estimation of prototype velocities. However, because this model was not density stratified and contained only one intake, some small variation in the prototype velocities can be expected under stratified flow conditions and when there is flow into adjacent intakes.

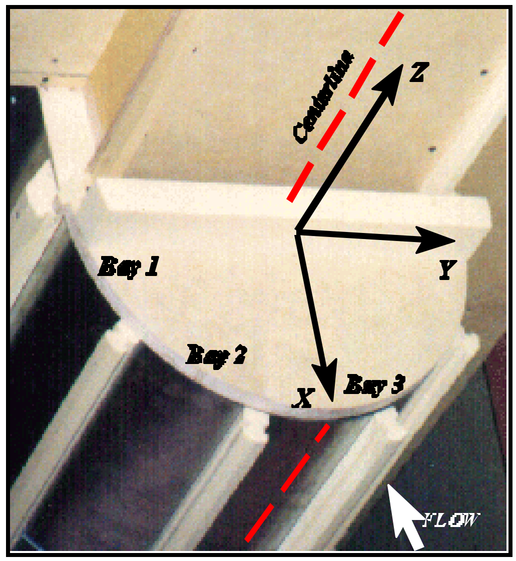

Figure 11.—Velocity profile coordinate system.

Water Hammer Tests

A series of tests were conducted to measure the dynamic (peak) pressures associated with a rapid gate closure to simulate a wicket gate closure in the prototype. Water hammer testing was not planned for in the original scope of work for the model study. The purpose of these tests was to collect data that could be compared with results from a numerical model which computes water hammer pressures for penstocks. A quick-closing butterfly valve was installed about 160 ft (prototype) downstream from the penstock intake. The valve was manually closed to model a 10-second (prototype) wicket gate closure rate. A 100 lb/in^2 Kistler dynamic pressure transducer (S/N C10178) was mounted in the trashrack structure floor. The transducer was flush mounted on the centerline of the intake structure, about 4.2 ft (prototype) from the face of the dam, or just upstream from the fixed-wheel gate slot. Because of model-prototype differences (e.g., pipe material, pipe length, pipe friction, and control of valve closure times) which significantly impact the magnitude of peak water hammer pressures, these data were considered qualitative and were not used for the design of the MLIS.

MODEL RESULTS

The following results were obtained from testing the 1:20 scale hydraulic model of a single penstock intake structure at Glen Canyon Dam.

Head Loss Versus Discharge

Head losses were evaluated over a discharge range of 1,000 to 4,000 ft^3/sec and at various water surface elevations, depending on the hydraulic model configuration.

The determination of the head loss for existing conditions allowed for the computation of the additional head loss associated with the selective withdrawal system modifications. Additional head losses attributed to the selective withdrawal system were obtained by subtracting the system head losses for existing condition from the system head losses for the modified structure. Uncertainties in the head loss data were determined by error analysis. The uncertainties reported were computed as the SDOM of maximum head loss measured. When the additional head loss was computed, the uncertainty was computed as the sum of the SDOMs from the two head loss measurements.

Existing conditions (modification zero) - Tests were conducted at reservoir elevations 3700, 3670, 3650, 3630, 3530, 3510, and 3490 ft. Discharges ranged from 1,000 to the maximum of 4,000 ft^3/sec. These test results showed that head loss was practically independent of water surface elevation (submergence), as is shown in figure 12. The maximum head loss for a discharge of 4,000 ft^3/sec was 3.8 ± 0.1 ft. The relationship that best describes the head loss versus discharge curve in figure 12 is expressed as equation 7. The coefficient of determination (R^2) for equation 7 is an indicator of the goodness of fit. A R^2 equal to 1.0 indicates perfect agreement between the data and the values computed using the best-fit equation. The system head loss coefficient (K_e) for this intake configuration was determined to be 0.30.

$$H_L = 8.298\times10^{-7} \cdot Q^{1.850} \quad with\ R^2 = 0.994 \qquad \textbf{(7)}$$

where: H_L is the head loss in ft
Q is the discharge in ft^3/sec
R^2 is the coefficient of determination of the best-fit equation

Selective withdrawal structure with the control gate in raised position and the addition of the stationary gate (modification two) - The same water surface elevations were tested as in modification zero to evaluate the additional head losses associated with cold water withdrawals through the modified intake. The results of these tests showed that the additional head losses were practically independent of water surface elevation, as is shown in figure 13. The relationship that best describes the additional system head loss versus discharge curve in figure 13 is expressed as equation 8. The maximum additional system head loss for a discharge of 4,000 ft^3/sec was 0.36 ± 0.11 ft. The additional system head loss coefficient (K_e') for this intake configuration was determined to be 0.03. The small amount of additional head loss was attributed to blocking 52 percent of the lower trashrack area and by adding a sharp-edged feature to the structure, namely the stationary gate.

$$Additional\ H_L = 3.139\times10^{-7} \cdot Q^{1.684} \quad with\ R^2 = 0.997 \qquad \textbf{(8)}$$

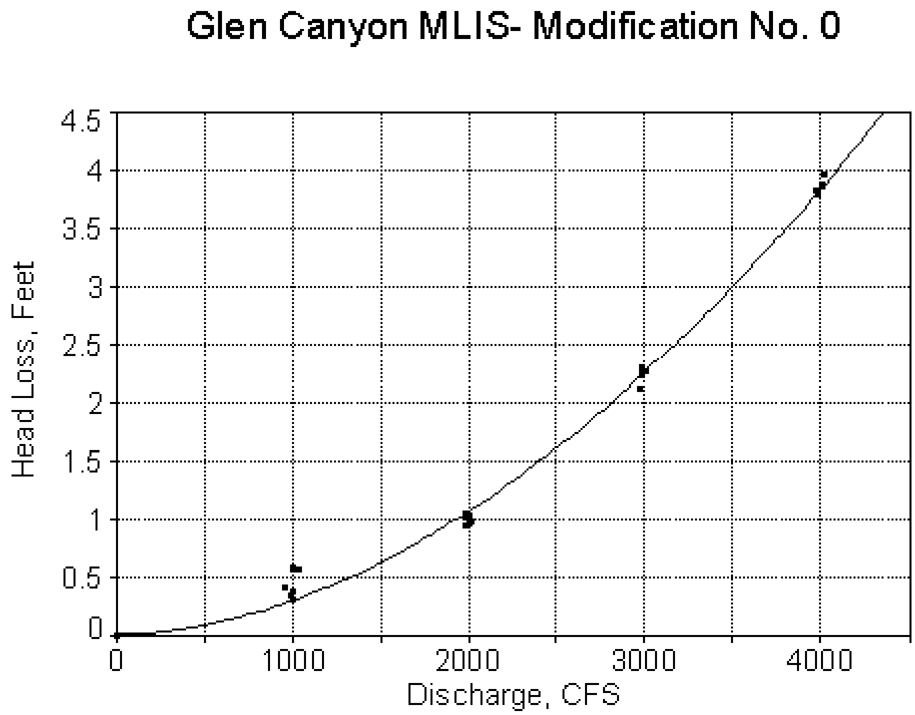

Figure 12.—Head loss as a function of discharge for existing penstock intake configuration (modification zero).

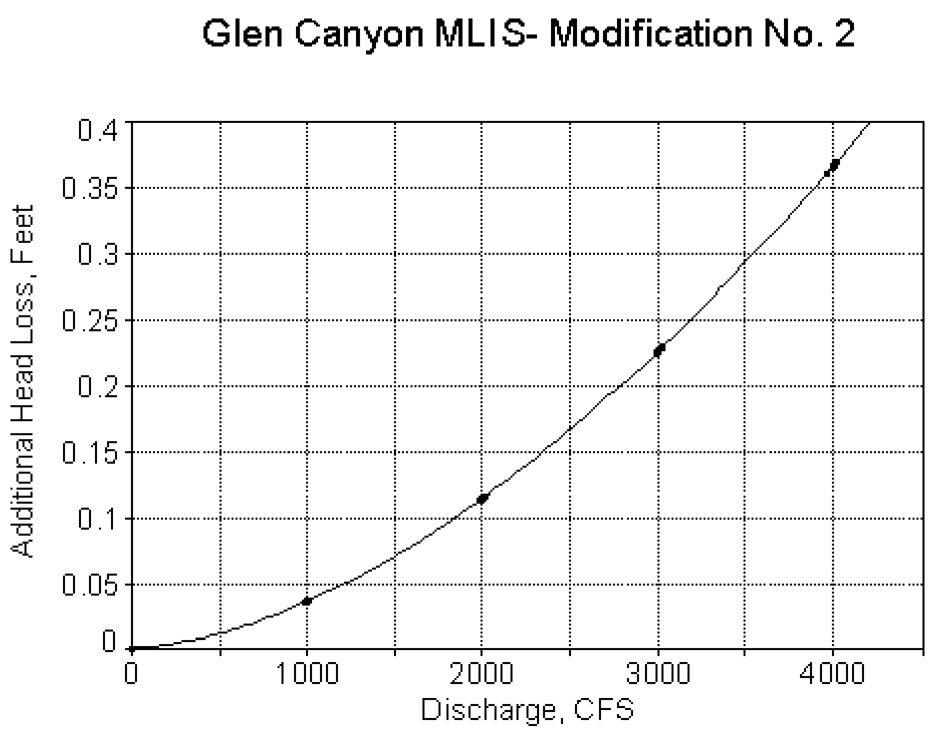

Figure 13.—Additional head loss as a function of discharge for cold water withdrawal through the MLIS (modification two).

Warm water withdrawal through a 20-ft-high trashrack with the control gate in down position (modification three) - Tests were conducted for water surface elevations 3650 to 3700 ft, every 10 ft, which represents a range of submergences from 20 to 70 ft. The flow range tested was from 1,000 to 4,000 ft^3/sec at 500 ft^3/sec increments. The results from this series of tests showed that the additional head loss was minimally affected by submergence levels, as shown in figure 14. The relationship that best describes the additional system head loss versus discharge curve in figure 14 is expressed as equation 9. The maximum additional head loss for a discharge of 4,000 ft^3/sec was 3.9 ± 0.1 ft. The additional system head loss coefficient (K_e') for this intake configuration was determined to be 0.31. The additional head loss was generated by flow over the sharp-crested relief gate, flow down the enclosed trashrack structure, and penstock entrance losses.

$$\textit{Additional}\ H_L = 7.188\times10^{-7} \cdot Q^{1.870} \quad \textit{with}\ R^2 = 0.997 \tag{9}$$

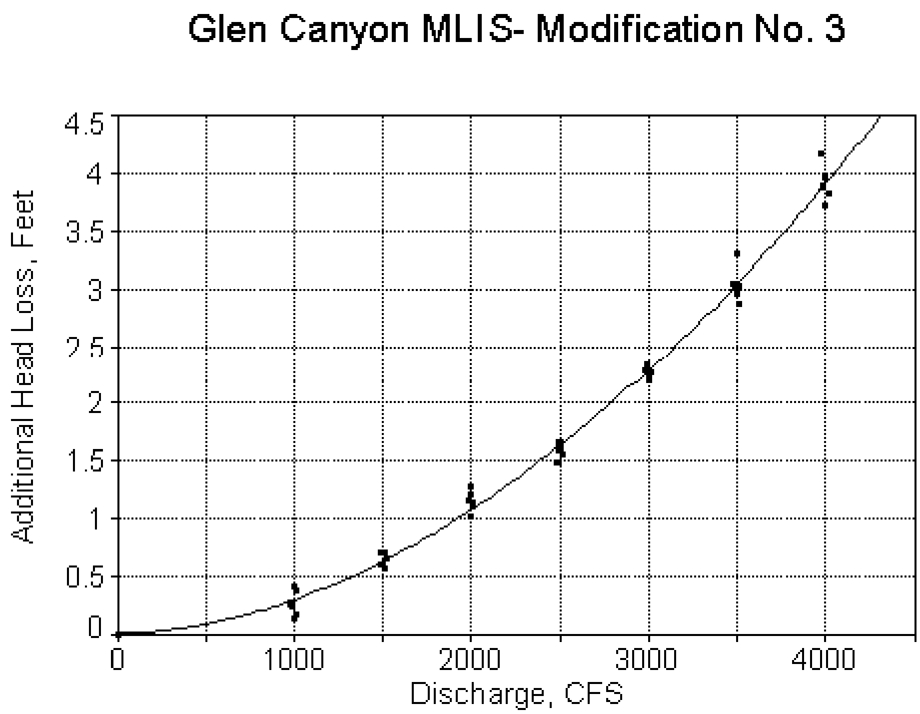

Figure 14.—Additional head loss for MLIS surface withdrawals (modification three).

Warm water withdrawal through a 20-ft-high trashrack with the control gate in down position and a solid trashrack lid (modification four) - Tests were conducted for water surface elevations 3660 to 3680, at 10-ft increments, which covers a range of submergences from 30 to 50 ft. The discharge range tested was from 1,000 to 4,000 ft^3/sec at 500 ft^3/sec increments. The results from this series of tests showed that the maximum additional head loss caused by adding the solid trashrack lid was about 0.25 ± 0.20 ft. This increase in additional head loss (compared to modification three) was caused by the reduction in trashrack's flow area. The relationship that best describes the additional system head loss versus discharge relationship is expressed as equation 10. The maximum additional head loss for a discharge of 4,000 ft^3/sec was 4.2 ± 0.2 ft. The additional system head loss coefficient (K_e') for this intake configuration was determined to be 0.32.

$$\text{Additional } H_L = 1.548\times10^{-7} \cdot Q^{2.063} \quad \text{with } R^2 = 0.999 \tag{10}$$

Selective withdrawal structure with the control gate in down position with a 25-ft-high trashrack and a solid lid (modification five) - Tests were conducted at water surface elevation 3670, or 40 ft of submergence. The flow range tested was from 2,000 to 4,000 ft^3/sec at 500 ft^3/sec increments. The maximum additional system head loss for a discharge of 4,000 ft^3/sec was 4.0 ± 0.2 ft. The relationship that best describes the additional head loss versus discharge curve in figure 15 is expressed as equation 11. The additional system head loss coefficient (K_e') for this intake configuration was determined to be 0.31.

$$\text{Additional } H_L = 7.781\times10^{-8} \cdot Q^{2.142} \quad \text{with } R^2 = 1.000 \tag{11}$$

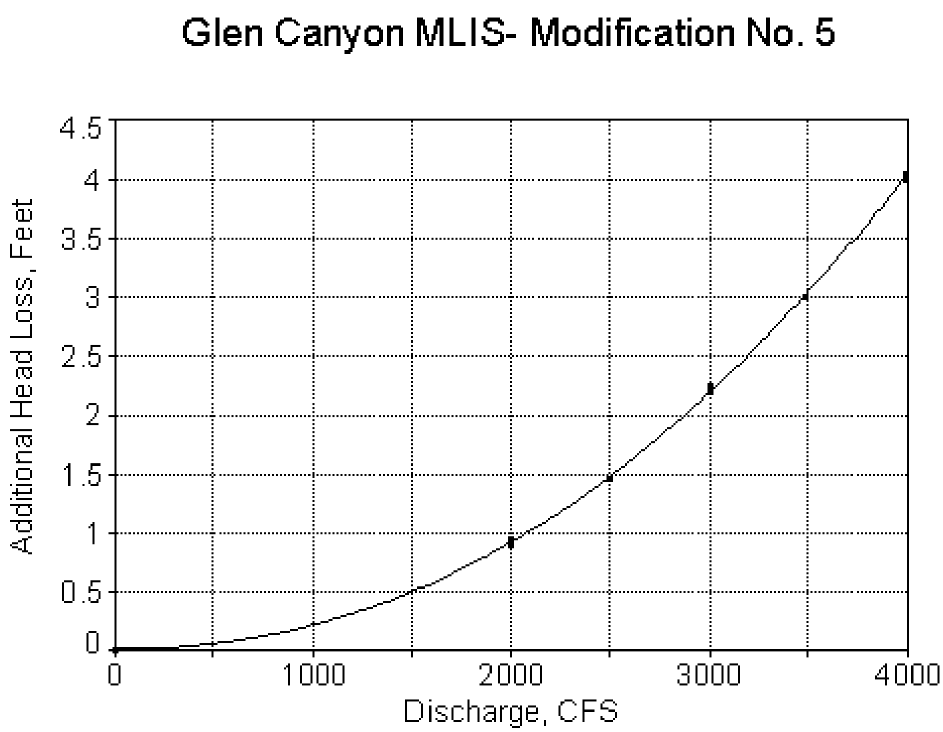

Figure 15.—Additional head loss for surface withdrawal through a 25-ft-high trashrack with a solid cover (modification five).

Same as modification five, except the control gate was raised 1 ft (modification six) - Tests were conducted for a 1-ft gate opening at water surface elevation 3670, or 40 ft of submergence. The discharge range tested was from 2,000 to 4,000 ft^3/sec at 500 ft^3/sec increments (figure 16). The relationship that best describes the additional head loss versus discharge curve in figure 16 is expressed as equation 12. The results from these tests showed that for partial control gate openings the head loss was reduced by about 1.2 ± 0.1 ft or 30 percent for the design discharge when compared to modification five. The maximum additional system head loss for a discharge of 4,000 ft^3/sec was 2.8 ± 0.1 ft. The additional system head loss coefficient (K_e') for this intake configuration was determined to be 0.21.

$$Additional\ H_L\ '\ 3.272\times10^{\&8}(Q^{2.202}\quad with\ R^2\ '\ 1.000 \qquad \textbf{(12)}$$

A partial gate opening is desirable for operations when surface withdrawals are initiated. An incremental gate opening procedure can be used to control the temperature increase in the tailwater. However, it is recommended that a minimum 1-ft gate opening be maintained to prevent vibration and hydraulic downpull on the control gates. These vibration and hydraulic downpull were not evaluated during model testing. Consequently, field evaluation of vibration is recommended.

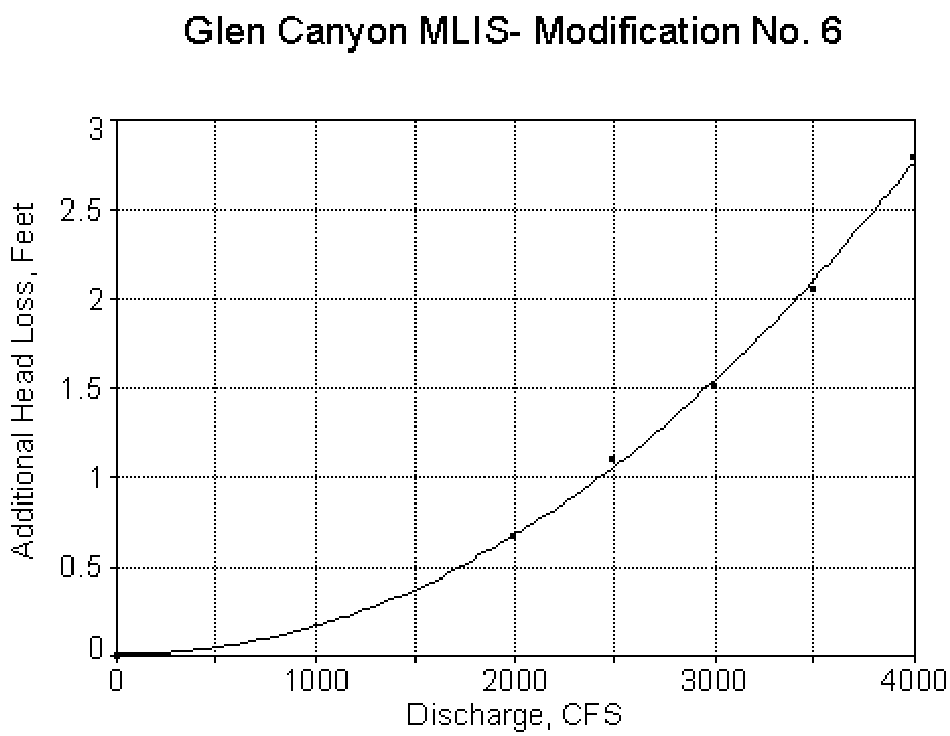

Figure 16.—Additional head loss for 1-ft opening between the stationary gate and the control gate (modification six).

Head loss across the control gate during surface withdrawals (modification seven) - Tests were conducted for water surface elevation 3670 to 3700 ft. The discharge range tested was 1,000 to 4,000 ft^3/sec at 500 ft^3/sec increments. The results from the tests showed that the head loss (differential) across the internal control gate was about 2.4 ± 0.1 ft at a discharge of 4,000 ft^3/sec (figure 17). For these head loss computations, the velocity head at the piezometer location was set equal to zero, which is a conservative assumption. When the internal velocity head loss at the top of the control gate, elevation 3500, is added (using velocity from table A5) to the piezometric head loss measured on the structure floor, the total head loss across the control gate increases by 1.4 ft, for a total of 3.8 ft. This combined head loss should be used in the design of the relief panel shear pins. The relationship that best describes the head loss versus discharge relationship is expressed as equation 13. A test was also run at a flow of 4,500 ft^3/sec to extend the relationship for future increases in discharge that may result from turbine upgrades. The head loss coefficient (K_e) between the reservoir and the floor of the trashrack structure was determined to be 0.19. This result indicates that about two-thirds of the additional system head loss occurs as water flows through the MLIS. The balance of the additional head loss occurs in the intake transition to the penstock.

$$H_L = 1.280\times10^{-7} \cdot Q^{2.018} \quad with\ R^2 = 0.999 \qquad \textbf{(13)}$$

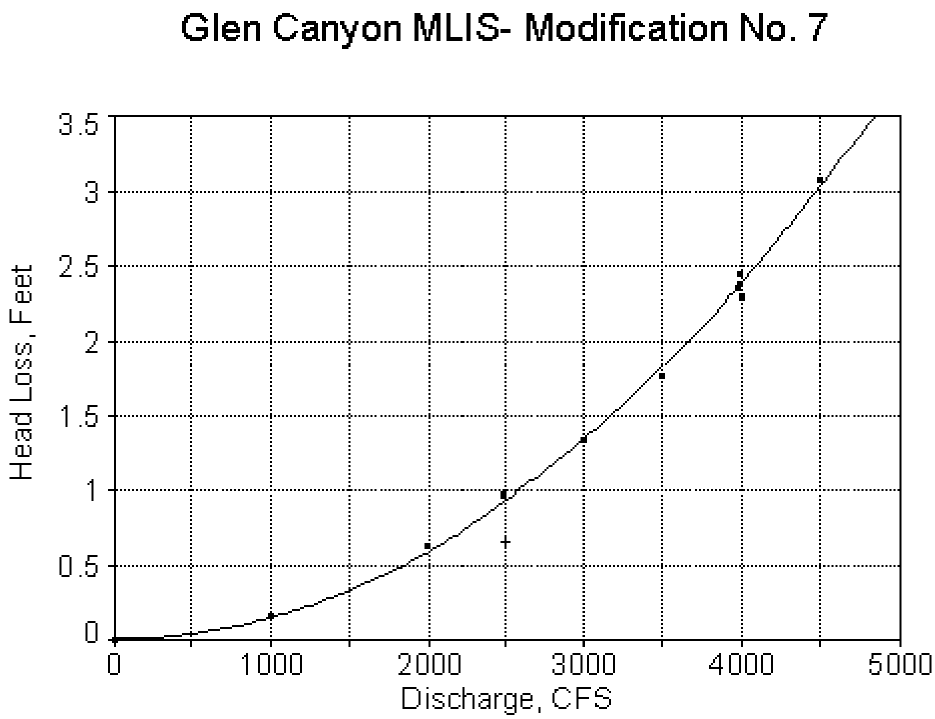

Figure 17.—Head loss across internal control gate (modification seven). Note: The data point indicated by a "+" was an outlier and was not used in the curve fit.

Model-Prototype Head Loss Comparison

As part of the water hammer evaluation, field tests were conducted to collect pressure measurements for a series of load rejection tests (Tolen 1999 and Cline 1999). These data were used to determine head losses in turbine units 3 and 4 for a wide range of discharges. The head loss data were adjusted to reduce pipe friction losses for a longer prototype penstock (680 ft) as compared to the model penstock which was 138 ft long. A comparison of model and prototype head losses is shown in figure 18. These data agree reasonably well, considering model limitations, such as differences in Reynolds number, penstock diameter, roughness coefficients, and intake shape. In addition, the pressure transducer was installed on the 6-inch diameter cooling water supply line which is located adjacent to the scroll case access door. The flow in this supply line was not measured; the velocity head in this pipe may account for the higher prototype head loss.

If an estimate of the total MLIS system head loss (from reservoir to turbine scroll case) is required, the prototype head losses in figure 18 can be added to the additional head losses calculated for any of the MLIS modifications tested in the model.

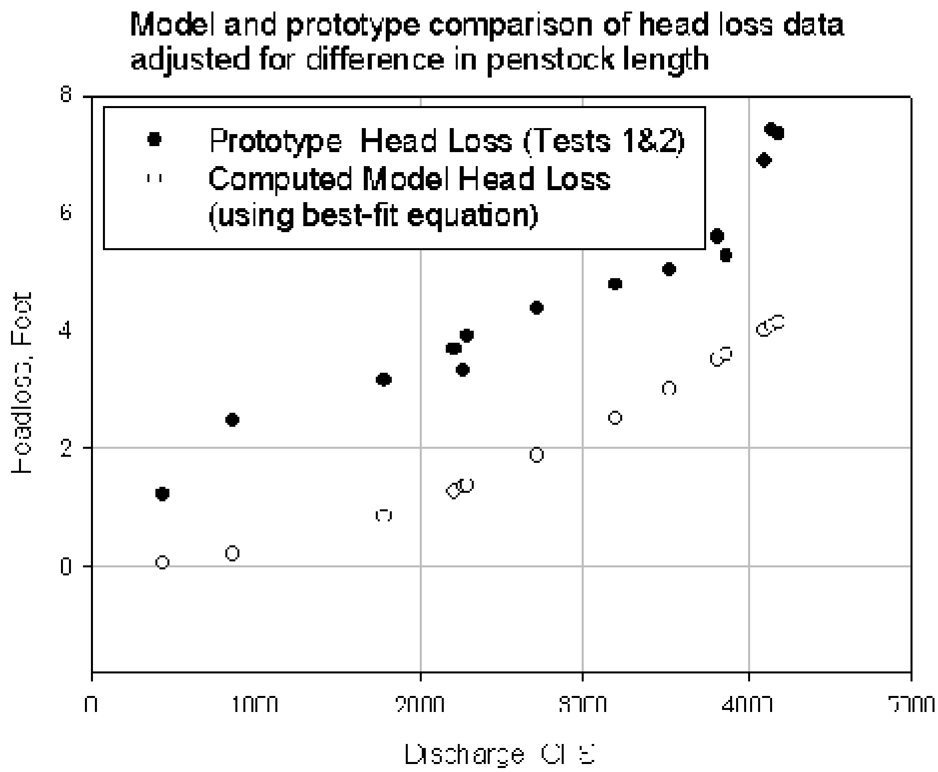

Figure 18.—Comparison plot of prototype and model head losses for the existing penstock intake configuration.

In conclusion, a summary of the system head loss curves for the existing, the MLIS cold water, and the MLIS warm water withdrawal configurations is shown in figure 19. This figure clearly shows the additional head loss associated with MLIS modifications when compared to the existing intake configuration.

Head loss versus submergence - Head losses through the MLIS were measured for submergences that varied from 20 to 70 ft and discharges that ranged from 1,000 ft^3/sec to 4,000 ft^3/sec. These data showed that submergence had a relatively small influence on the system head loss. For example, for a flow of 4,000 ft^3/sec the K_e values varied from 0.62 to 0.60 for submergences of 20 and 40 ft, respectively. This result indicates that head losses through the new trashrack were small and did not vary much with change in approach velocities. However, the submergence does influence the strength and frequency of vortices which form near the intake.

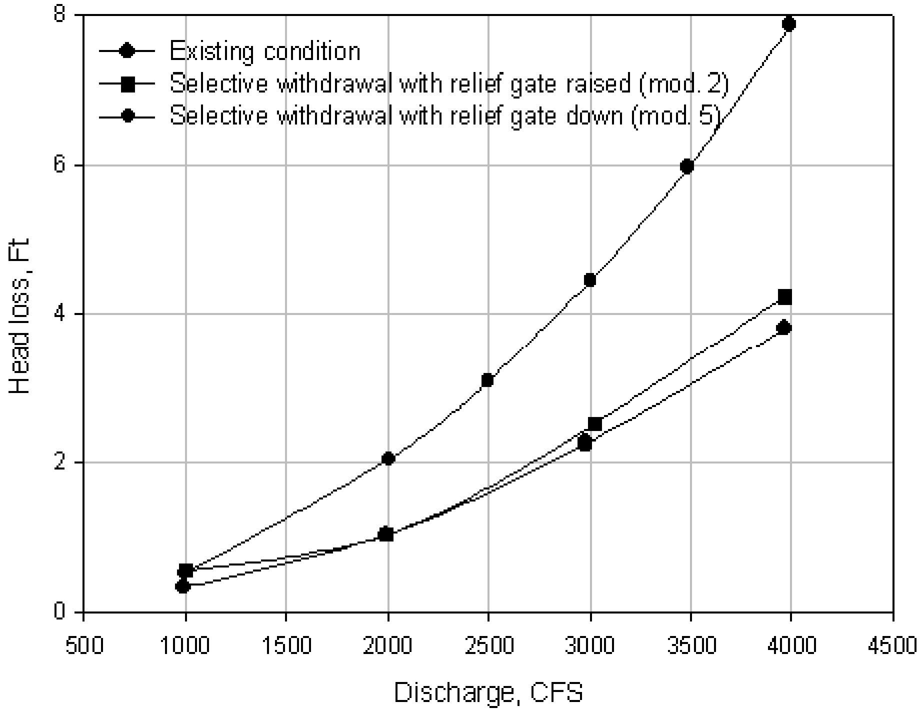

Figure 19.—Comparison of the system head loss curves for the existing, the MLIS cold water, and the MLIS warm water withdrawal configurations.

Vortex Formation Potential

Physical modeling has inherent similitude problems that make predicting vortex strength a qualitative evaluation. A detailed description of these modeling limitations can be found in a Reclamation report entitled *Hydraulic Model Vortex Study - Grand Coulee Third Powerplant* (Zeigler 1976). Dhillon (1979) states, "If a 1:20 scale model satisfies the Reynolds number criterion: $\Re \geq 3\times10^4$, there is a good agreement of vortex strength between model and prototype when operated at Froude-scaled flow." The Reynolds numbers tested in this study met or exceeded this criterion and are shown in table 2.

For this model, the Reynolds number describing the MLIS is defined as:

$$\Re_{model} = \frac{VD}{\nu}$$

where: V is the velocity inside the MLIS in ft/sec
D is the control gate diameter, 0.58 ft
$\nu = 1.22\times10^{-5}$ ft^2/sec - the kinematic viscosity of the water at T = 60°F.

Table 2.—Reynolds Numbers (ú) for a Range of Flows in the Glen Canyon MLIS Model (1:20 scale)

Discharge (ft^3/sec)	2,000	3,000	4,000
$ú_{model}$	6.5×10^4	8.6×10^4	1.18×10^5

Hecker (1981) reported that scale effects are negligible when evaluating vortices smaller than type 5 in a model meeting the above Reynolds number criterion. However, air core vortices in the model will likely be stronger in the prototype due to differences in viscous and surface tension forces. Hecker outlined several other factors which can contribute to model-prototype differences in vortex formation, they are:

1. Poor representation of approach topography and boundary roughness in the model because vorticity approaching the intake has a first-order effect on the resulting vortex strength.

2. Viscous scale effects on modeling flow features such as trash racks, baffles, and vortex suppressors.

3. Small prototype differences in topographic or structural changes from those tested in the model. For example, construction roads, cut and fill areas, modified approach channels or wing walls.

4. Wind induced currents or modified flow patterns associated with spillway operations.

5. Ambient density stratification.

For this model study, items 2, 4, and 5 are most likely to affect vorticity similitude.

In the report, *Model-Prototype Comparison of Free Surface Vortices* (Hecker 1981), it was reported that at near minimum submergence, two prototype vortices developed within the existing intake structure at Glen Canyon Dam. The observed vortices were 1 ft in diameter and had a 6-ft core depth. Type 3 vortices were also observed in the hydraulic model for existing conditions at water surface elevations between 3490 and 3530 ft at a discharge of 4,000 ft^3/sec (figure 20). However, the model vortices were not observed at the same locations as the prototype, but it was difficult to make observations inside the trashrack structure.

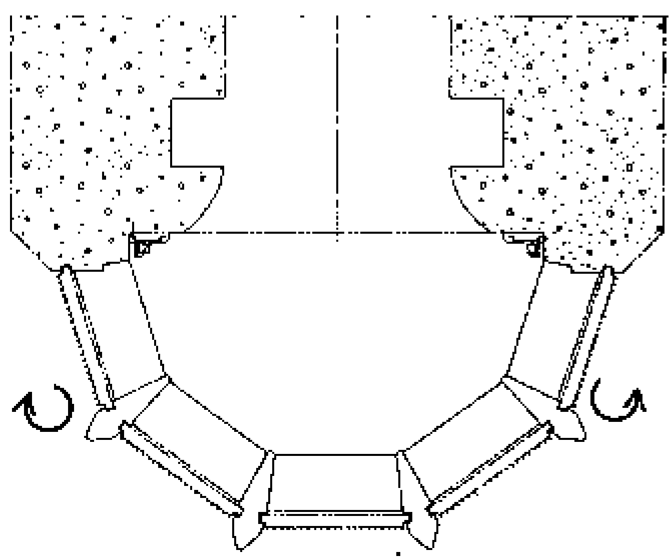

Figure 20.—Location of vortices (type 3) which formed for cold water withdrawal at water surface elevation 3490 (20 ft of submergence).

Flow visualization techniques were used to evaluate vortex formation potential of the MLIS. For the MLIS configured with the control gate in a raised position at the design discharge and at water surface elevation of 3490 ft, two type 3 vortices were observed just outside the trashrack structures (figure 20).

For the MLIS configured with the control gate in a down position, vortices were observed at submergences ranging from 20 to 50 ft over a wide range of discharges. During tests of modification three, the location of the vortices were near the stop-log guides on both sides of the intake structure (figure 21).

Table 3 summarizes the vortices observed for various hydraulic conditions at water surface elevations between 3650 and 3680 ft. In table 3, symmetric means two vortices which formed at similar locations symmetric about the structure centerline, but not necessarily at the same time. Likewise, transient vortices formed at random locations and they would dissipate a short time after they developed.

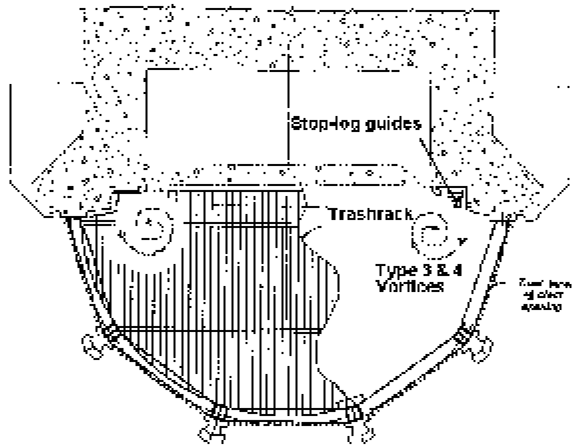

Figure 21.—Location of typical vortices for the trashrack with a porous lid.

Table 3.—Summary of Vortex Locations and Strength for the MLIS Surface Withdrawals Vortex Strength is a Function of Submergence and Discharge

<table>
<tr><th>WSEL</th><th>3680 Ft</th><th>3670 Ft</th><th>3660 Ft</th><th>3650 Ft</th></tr>
<tr><td>Submergence</td><td>50 Ft</td><td>40 Ft</td><td>30 Ft</td><td>20 Ft</td></tr>
<tr><td>Discharge</td><td></td><td></td><td></td><td></td></tr>
<tr><td>4000 ft³/sec</td><td>Type 2, transient</td><td rowspan="3">Symmetric vortices - type 3, transient</td><td rowspan="3">Symmetric vortices - type 3 and 4, permanent</td><td rowspan="2">Symmetric vortices - type 5, permanent</td></tr>
<tr><td>3500 ft³/sec</td><td rowspan="2">Type 1, transient</td></tr>
<tr><td>3000 ft³/sec</td><td rowspan="3">Type 4, permanent</td></tr>
<tr><td>2500 ft³/sec</td><td rowspan="4">None</td><td rowspan="4">Type 2, transient</td><td rowspan="2">Type 2, permanent</td></tr>
<tr><td>2000 ft³/sec</td></tr>
<tr><td>1500 ft³/sec</td><td rowspan="2">Type 1, permanent</td><td rowspan="2">Type 3, permanent</td></tr>
<tr><td>1000 ft³/sec</td></tr>
</table>

Vortex Suppression Tests - Vortex suppression features were studied in the model in an effort to keep submergence limits to a minimum for MLIS operations. The following vortex suppression systems were tested:

- A lower porosity trashrack lid

- A solid, floating, semi-circular raft with the same radius as the trashrack structure

- Two-ft-, four-ft-, and six-ft-wide rectangular panels which covered a portion of the trashrack closest to the stop-log guides (figure 22)

- A solid trashrack lid (figures 23 and 24)

The solid trashrack lid performed the best of the four alternatives tested. However, the solid trashrack lid reduced the total trashrack area by 360 ft^2, or 30 percent. To replace this lost area, the trashrack height was changed from 20 to 25 ft, which added 210 ft^2 of trashrack area.

Vortex strengths observed for the 25-ft-high trashrack with a solid lid (figure 24) were weaker than those which occurred for the original trashrack design. In addition, vortices did not form inside the trashrack perimeter. All observed vortices formed outside the trashrack perimeter and would quickly dissipate if they moved inside the trashrack perimeter. It should be noted that there is some uncertainty in the reported vortex strengths because the model consisted of only one intake structure. Interaction between adjacent intakes may worsen or lessen the vortex strength in a prototypical situation.

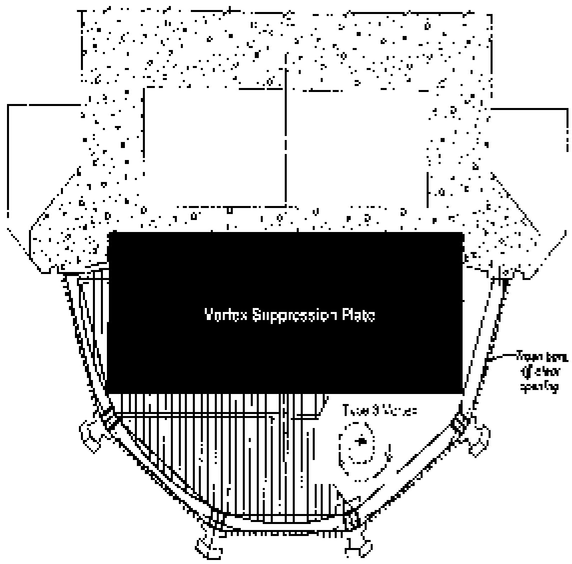

Figure 22.—Example of vortex suppression plate.

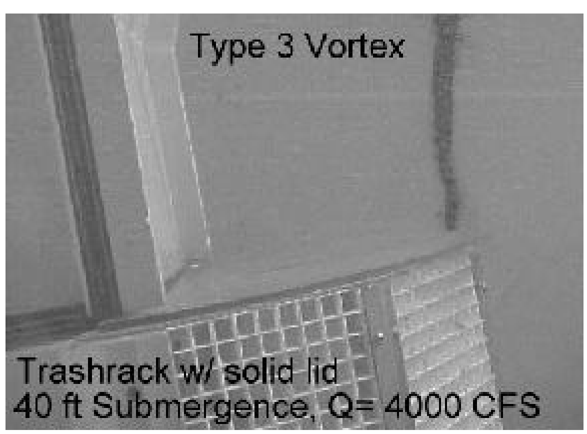

Figure 23.—Photograph of vortex dye core after a solid lid was installed on the trashrack. Unlike the porous lid, these vortices were intermittent and the location of vortex formation was unpredictable.

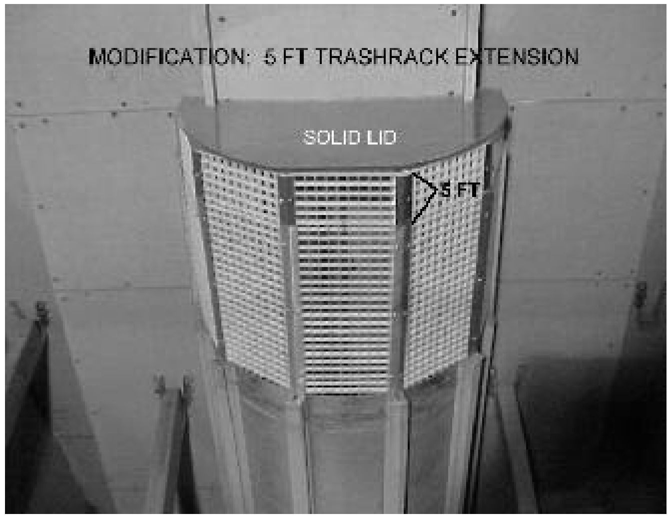

Figure 24.—Final configuration of 25-ft-high MLIS trashrack structure with a solid lid.

Velocity Profiles

Several internal and external velocity profiles were collected for surface and low-level withdrawals. All internal and external velocity profiles were measured along the trashrack structures centerline 8 ft upstream from the fixed-wheel gate slot and 6.7 ft upstream from the trashracks, respectively. In general, internal velocities were highly three dimensional, and external velocities were nearly horizontal. For presentation purposes, figures containing velocity profiles show the velocity magnitudes. The x, y, and z velocity components are presented in tables in the appendix.

For MLIS cold water withdrawals (modification two), external and internal velocity profiles were measured at the trashrack structure for the design discharge of 4,000 ft^3/sec and a water surface elevation of 3630 ft. Figure 25 illustrates that the internal velocity decreases proportionally with elevation, from a maximum velocity of 9.2 ft/sec near the stationary gate, to 0.2 ft/sec at elevation 3500 ft. For the same hydraulic conditions, the external velocities are five times smaller than the internal velocities.

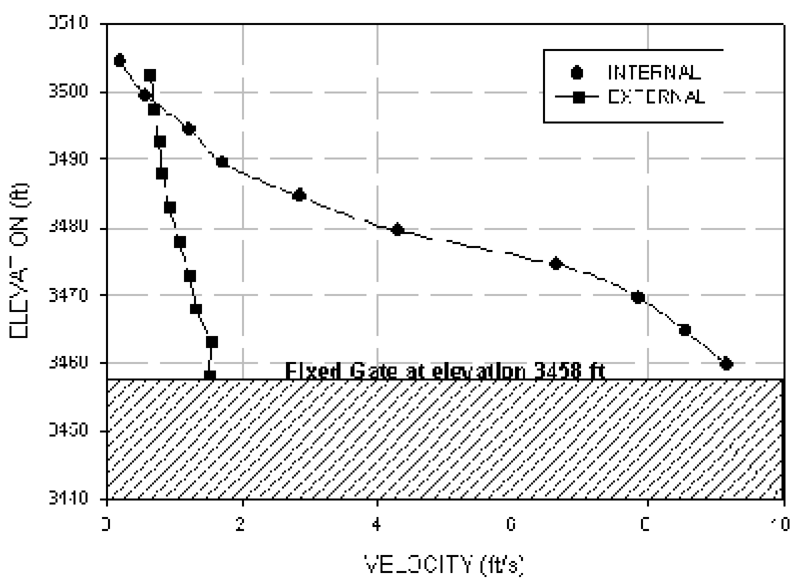

Figure 25.—Internal and external velocity profiles collected for cold water withdrawal (modification two). The flow was 4,000 ft^3/sec, and the reservoir water surface elevation was 3630.

For warm water withdrawals (modification three), internal velocity profiles were measured at reservoir elevation 3700 for three discharges: 4,000; 3,000; and 2,000 ft^3/sec (figure 26). These velocity profiles have a similar shape, and the maximum velocity, 11.1 ft/sec, was measured near elevation 3470 for a discharge of 4,000 ft^3/sec. Similarly, figure 27 shows the external velocity profiles collected for three different discharges. The maximum velocity, 5.2 ft/sec, was measured at elevation 3635, 7 ft above the crest of the uncontrolled intake.

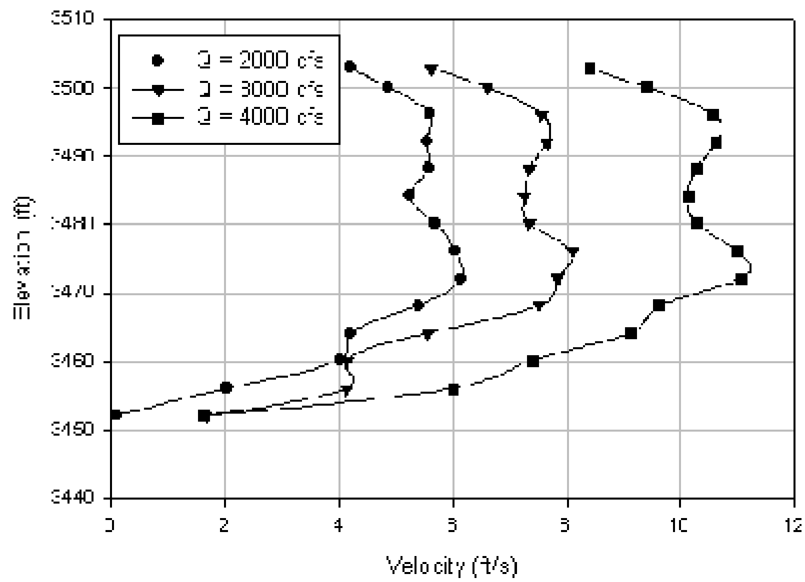

Figure 26.—Comparison of internal velocity profiles for warm water withdrawal (modification three) at water surface elevation 3700. Velocities were measured 8 ft upstream from face of the fixed wheel gate slot.

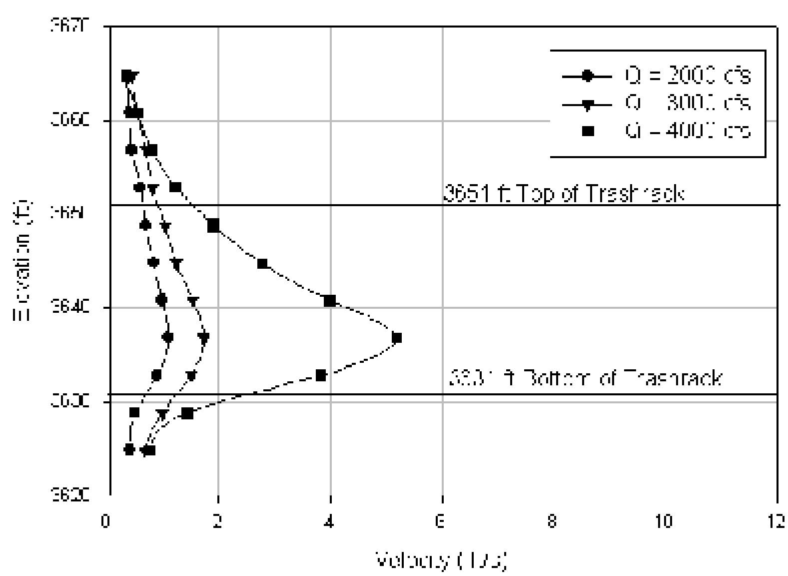

Figure 27.—Comparison of external velocity profiles for warm water withdrawal (modification three) at water surface elevation 3700. Velocities were measured 6.7 ft upstream from face of trash rack.

Figure 28 shows two external velocity profiles collected along the center line of bay 3, for a 1-ft gate opening in the control gate (modification six). The velocity profiles were measured during discharges of 3,000 and 4,000 ft^3/sec. A maximum velocity of about 1.5 ft/sec was observed at elevation 3465.

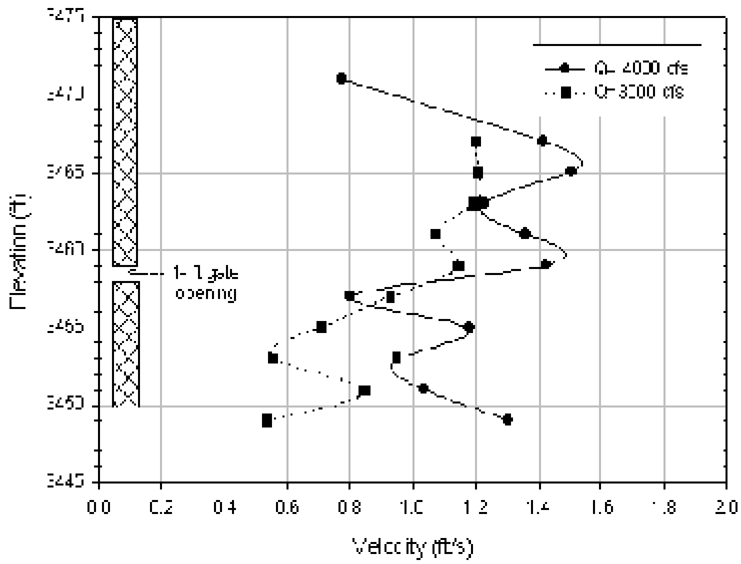

Figure 28.—Comparison of velocity profiles collected 6 ft upstream from the trashrack for two discharges. The control gate was set with a 1-ft gate opening (modification six).

These model velocity profiles are useful from a design standpoint, but they do not describe the velocity profiles which would develop in a stratified reservoir. Research by Bohan and Grace (1973) resulted in empirical equations which describe the withdrawal limits and velocity profiles which develop for a submerged intake near the surface of a stratified reservoir. These equations are used in a U.S. Army Engineers Waterways Experiment Station model called SELECT (USAEWES 1987). This model can be used to determine the withdrawal limits and velocity profiles for a given reservoir temperature profile. For example, if the MLIS were operated for the same flow and water surface elevations, but with two different reservoir stratifications, the velocity profiles in the withdrawal zone would differ. Figure 29 shows the velocity profiles computed using the SELECT model and field measured temperature profiles for April and September 1992. Both model runs were made with a flow of 4,000 ft^3/sec and 40 ft of submergence on the MLIS intake. The maximum approach velocity is the same (5.2 ft/sec), but the vertical extent of the withdrawal zone becomes narrower for a stronger reservoir stratification. The SELECT computed release temperatures for the April and September withdrawals were 51.5 and 74.1EF, respectively.

The SELECT model can be a useful tool for project operators when they need to know what the release temperature through the MLIS would be for a known flow, reservoir elevation, and temperature profile. I recommend developing a calibrated SELECT model for the Glen Canyon MLIS to assist in planning MLIS operations.

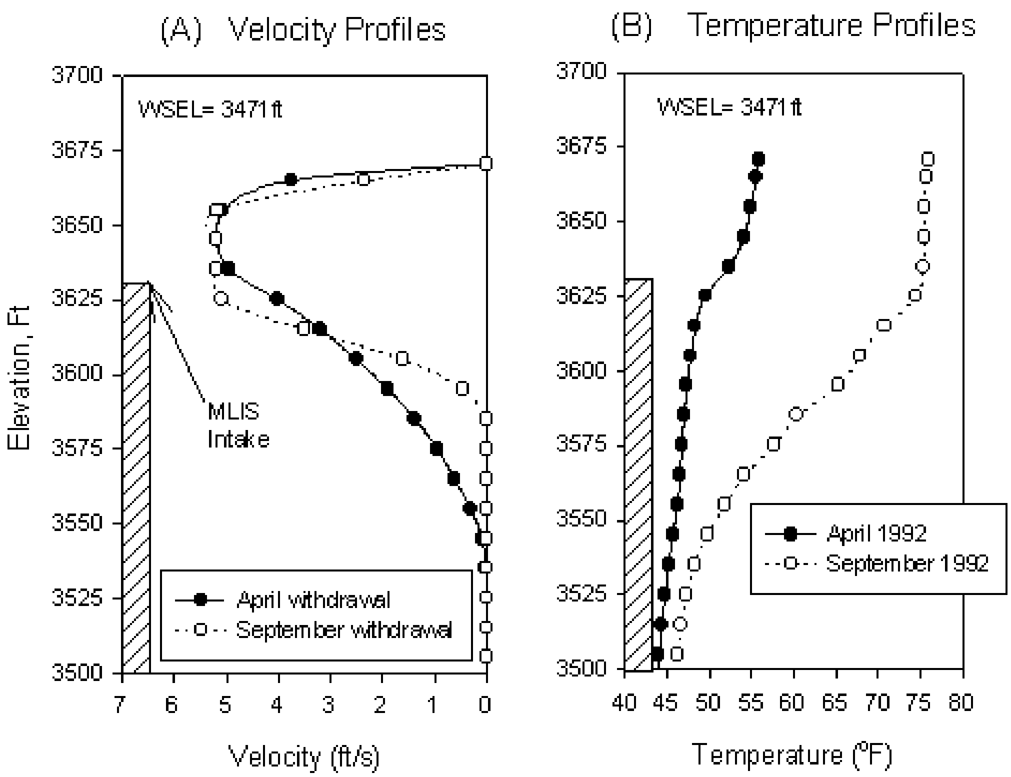

Figure 29.—(A) Velocity profiles for April and September surface withdrawals through the MLIS.
(B) Forebay temperature profiles for April and September 1992.

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APPENDIX

Summary of Velocity Profile Data

Table A1. External velocity profile collected during testing of modification 2. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3630 feet.

Elevation	**Vx-Avg**	**Vy-Avg**	**Vz-Avg**	**Vmag**	**V/horiz**
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3448.3	0.17	-0.78	0.89	1.20	0.80
3453.2	-0.75	-0.91	0.77	1.41	1.18
3458.2	-1.11	-0.64	0.83	1.52	1.28
3463.1	-1.43	-0.26	0.48	1.53	1.46
3468.0	-1.19	0.21	0.50	1.31	1.21
3472.9	-1.18	0.21	0.22	1.22	1.20
3477.9	-1.03	0.23	-0.01	1.06	1.06
3482.8	-0.86	0.27	-0.15	0.91	0.90
3487.7	-0.80	0.02	-0.13	0.81	0.80
3492.7	-0.70	-0.07	-0.32	0.78	0.71
3497.6	-0.59	0.01	-0.35	0.68	0.59
3502.5	-0.58	0.04	-0.29	0.65	0.58

Table A2. Internal velocity profile collected during testing of modification 2. The profile was collected 8 feet upstream from the fixed-wheel gate slot. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3630 feet.

Elevation	**Vx-Avg**	**Vy-Avg**	**Vz-Avg**	**Vmag**	**V/horiz**
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3459.7	-8.08	-3.70	-2.27	9.17	8.89
3464.6	-7.91	-2.37	-2.31	8.58	8.26
3469.6	-6.98	-2.30	-2.82	7.88	7.35
3474.5	-5.60	-1.25	-3.36	6.65	5.74
3479.5	-2.70	-0.69	-3.29	4.32	2.79
3484.4	-2.10	-0.16	-1.94	2.87	2.11
3489.4	-1.54	-0.13	-0.75	1.72	1.55
3494.3	-1.20	-0.25	0.13	1.23	1.23
3499.3	-0.56	0.01	0.15	0.58	0.56
3504.2	-0.09	0.06	-0.18	0.21	0.11

Table A3. Internal velocity profile collected during testing of modification 3. The profile was collected 8 feet upstream from the fixed wheel gate slot. The discharge was 2,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3452.1	-0.02	-0.09	0.01	0.10	0.09
3456.1	-1.34	-0.92	-1.20	2.02	1.63
3460.1	-2.48	-0.88	-3.02	4.01	2.63
3464.1	-3.02	-0.38	-2.89	4.20	3.04
3468.1	-3.20	-0.53	-4.30	5.38	3.24
3472.1	-3.02	-1.38	-5.18	6.15	3.32
3476.1	-2.62	-0.55	-5.43	6.05	2.68
3480.1	-1.52	-0.31	-5.48	5.69	1.55
3484.1	-0.81	-0.32	-5.16	5.24	0.87
3488.1	-0.28	0.07	-5.57	5.58	0.29
3492.1	-0.21	-0.05	-5.53	5.54	0.22
3496.1	-0.28	0.15	-5.58	5.59	0.32
3500.1	-0.27	0.03	-4.86	4.87	0.27
3502.7	-0.09	0.06	-4.18	4.18	0.11

Table A4. Internal velocity profile collected during testing of modification 3. The profile was collected 8 feet upstream from the fixed wheel gate slot. The discharge was 3,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3452.1	-1.26	-1.08	0.01	1.65	1.65
3456.1	-2.84	-1.84	-2.36	4.13	3.38
3460.1	-3.36	-0.71	-2.29	4.12	3.43
3464.1	-3.72	-1.31	-3.90	5.55	3.94
3468.1	-4.24	-1.01	-6.11	7.51	4.36
3472.1	-4.05	-0.58	-6.69	7.84	4.09
3476.1	-3.48	-1.15	-7.22	8.10	3.66
3480.1	-2.09	-0.72	-6.99	7.33	2.21
3484.1	-0.69	0.12	-7.23	7.26	0.70
3488.1	-0.21	-0.04	-7.33	7.33	0.21
3492.1	0.30	0.25	-7.63	7.64	0.40
3496.1	-0.02	0.10	-7.54	7.55	0.11
3500.1	-0.36	0.19	-6.58	6.60	0.41
3502.7	-0.11	0.04	-5.62	5.62	0.12

Table A5. Internal velocity profile collected during testing of modification 3. The profile was collected 8 feet upstream from the fixed wheel gate slot. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3452.1	-0.82	1.42	-0.02	1.64	1.64
3456.1	-4.58	-1.61	-3.53	6.00	4.86
3460.1	-4.73	-2.63	-5.01	7.38	5.42
3464.1	-5.61	-2.54	-6.75	9.14	6.16
3468.1	-5.22	-2.87	-7.58	9.64	5.96
3472.1	-5.07	-2.96	-9.39	11.07	5.87
3476.1	-4.53	-0.76	-9.99	10.99	4.59
3480.1	-2.70	-0.54	-9.92	10.30	2.75
3484.1	-1.22	-0.62	-10.04	10.13	1.37
3488.1	-0.30	-0.24	-10.28	10.29	0.38
3492.1	0.01	-0.06	-10.64	10.64	0.06
3496.1	-0.44	-0.06	-10.56	10.57	0.44
3500.1	-0.68	-0.24	-9.36	9.39	0.72
3502.7	-0.29	-0.17	-8.39	8.40	0.33

Table A6. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3622.8	-0.38	-0.16	0.77	0.75	0.41
3626.8	-0.55	-0.03	1.06	1.44	0.55
3630.8	-1.48	-0.22	1.25	3.81	1.50
3634.8	-2.16	-0.11	0.70	5.18	2.17
3638.8	-2.00	0.06	0.00	4.00	2.00
3642.8	-1.64	-0.12	-0.30	2.78	1.64
3646.8	-1.37	0.02	-0.15	1.90	1.37
3650.8	-1.08	-0.03	-0.24	1.23	1.08
3654.8	-0.85	0.10	-0.28	0.80	0.85
3658.8	-0.64	0.14	-0.34	0.54	0.65
3662.8	-0.51	0.11	-0.26	0.34	0.52

Table A7. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 2,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
feet	feet/s	feet/s	feet/s	feet/s	ft/sec
3622.8	-0.24	-0.06	0.33	0.41	0.24
3626.8	-0.24	-0.18	0.41	0.50	0.30
3630.8	-0.76	0.00	0.49	0.90	0.76
3634.8	-1.08	-0.06	0.20	1.10	1.08
3638.8	-0.98	0.00	-0.12	0.99	0.98
3642.8	-0.76	0.03	-0.35	0.83	0.76
3646.8	-0.57	-0.09	-0.39	0.70	0.58
3650.8	-0.44	-0.05	-0.40	0.60	0.45
3654.8	-0.26	0.02	-0.37	0.45	0.26
3658.8	-0.28	0.09	-0.27	0.40	0.30
3662.8	-0.05	-0.01	-0.34	0.34	0.05

Table A8. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 3,000 ft^3/sec and the reservoir elevation was 3700 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
feet	feet/s	feet/s	feet/s	feet/s	ft/sec
3622.8	-0.19	-0.11	0.62	0.66	0.22
3626.8	-0.54	-0.14	0.81	0.98	0.56
3630.8	-1.15	-0.24	0.94	1.50	1.18
3634.8	-1.64	-0.19	0.53	1.74	1.65
3638.8	-1.54	-0.11	0.02	1.54	1.54
3642.8	-1.23	-0.07	-0.22	1.25	1.23
3646.8	-0.93	0.23	-0.40	1.04	0.95
3650.8	-0.76	-0.11	-0.28	0.82	0.77
3654.8	-0.59	0.16	-0.32	0.69	0.61
3658.8	-0.47	-0.01	-0.27	0.54	0.47
3662.8	-0.33	-0.08	-0.29	0.45	0.34

Table A9. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3670 feet.

Elevation ft	Vx-Avg ft/sec	Vy-Avg ft/sec	Vz-Avg ft/sec	Vmag ft/sec	V/horiz ft/sec
3622.8	-0.31	-0.03	0.88	0.94	0.31
3626.8	-0.72	0.00	1.26	1.45	0.72
3630.8	-1.60	-0.04	1.26	2.04	1.60
3634.8	-2.17	-0.11	0.76	2.31	2.18
3638.8	-2.07	0.06	0.25	2.09	2.07
3642.8	-1.68	0.04	-0.04	1.68	1.68
3646.8	-1.39	0.06	-0.35	1.43	1.39
3650.8	-1.10	0.23	-0.33	1.17	1.12
3654.8	-0.89	0.15	-0.23	0.93	0.90
3658.8	-0.85	0.10	-0.18	0.88	0.86
3662.8	-0.81	0.07	-0.02	0.81	0.81

Table A10. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3660 feet.

Elevation ft	Vx-Avg ft/sec	Vy-Avg ft/sec	Vz-Avg ft/sec	Vmag ft/sec	V/horiz ft/sec
3622.8	-0.43	0.02	1.32	1.39	0.43
3626.8	-1.26	0.14	1.92	2.30	1.26
3630.8	-2.59	0.08	1.81	3.16	2.59
3634.8	-2.83	0.25	0.66	2.92	2.84
3638.8	-2.33	0.25	0.15	2.35	2.35
3642.8	-1.90	0.25	-0.21	1.93	1.92
3646.8	-1.51	0.30	-0.16	1.55	1.54
3650.8	-1.34	0.27	-0.05	1.36	1.36
3653.8	-1.29	0.19	-0.04	1.31	1.31

Table A11. External velocity profile collected during testing of modification 3. The profile was collected 6.7 feet upstream from the bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3650 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3610.8	-0.10	-0.11	0.72	0.74	0.15
3614.8	-0.12	0.03	0.92	0.93	0.13
3618.8	-0.23	0.08	1.12	1.15	0.24
3622.8	-0.36	0.02	1.58	1.62	0.36
3626.8	-1.21	0.06	2.23	2.54	1.21
3630.8	-2.79	-0.19	2.21	3.57	2.80
3634.8	-3.40	-0.23	1.04	3.57	3.41
3638.8	-3.04	0.08	0.63	3.10	3.04
3640.8	-2.94	0.29	0.64	3.02	2.96

Table A12. External velocity profile collected during testing of modification 6 (1-foot relief gate opening). The profile was collected 6.7 feet upstream from the lower bay 3 trashrack. The discharge was 3,000 ft^3/sec and the reservoir elevation was 3670 feet.

Elevation	Vx-Avg	Vy-Avg	Vz-Avg	Vmag	V/horiz*
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3449.0	0.04	-0.11	0.02	0.12	0.12
3451.0	0.17	-0.04	0.08	0.19	0.17
3453.0	0.05	-0.11	0.02	0.12	0.12
3455.0	-0.06	-0.15	0.00	0.16	0.16
3457.0	-0.15	-0.08	-0.11	0.21	0.17
3459.0	-0.02	0.13	0.22	0.26	0.13
3461.0	-0.13	0.16	-0.13	0.24	0.20
3463.0	-0.25	0.01	-0.09	0.27	0.25
3465.0	-0.18	0.02	-0.20	0.27	0.18
3467.0	-0.12	0.10	-0.22	0.27	0.15

* The ADV probe was misaligned with respect to the intake centerline, so the horizontal resultant of the Vx-Avg and Vy-Avg components should be considered as the best estimate of the approach velocity.

Table A13. External velocity profile collected during testing of modification 6 (1-foot relief gate opening). The profile was collected 6.7 feet upstream from the lower bay 3 trashrack. The discharge was 4,000 ft^3/sec and the reservoir elevation was 3670 feet.

Elevation	**Vx-Avg**	**Vy-Avg**	**Vz-Avg**	**Vmag**	**V/horiz***
ft	ft/sec	ft/sec	ft/sec	ft/sec	ft/sec
3449.0	0.28	-0.05	0.06	0.29	0.28
3451.0	-0.02	-0.10	0.21	0.23	0.10
3453.0	0.18	-0.10	0.05	0.21	0.21
3455.0	0.09	-0.21	0.13	0.26	0.23
3457.0	0.03	-0.10	0.14	0.18	0.11
3459.0	-0.21	-0.18	0.15	0.32	0.28
3461.0	-0.16	0.20	-0.17	0.30	0.25
3463.0	-0.26	-0.09	-0.04	0.27	0.27
3465.0	-0.29	-0.09	-0.15	0.34	0.30
3467.0	-0.26	-0.03	-0.18	0.32	0.26
3471.0	-0.17	-0.05	-0.01	0.17	0.17

* The ADV probe was misaligned with respect to the intake centerline, so the horizontal resultant of the Vx-Avg and Vy-Avg components should be considered as the best estimate of the approach velocity.