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HYDRAULIC SEDIMENT MODEL STUDY FOR PROPOSED BLANCO DIVERSION DAM MODIFICATION

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16. ABSTRACT

A 1/16-scale sediment model was used to study proposed modifications of Blanco Diversion Dam, located on a steep mountain stream. Pit-run sand was used to simulate bed sediment sizes ranging from cobbles to fine sand, including sizes in the coarsest 7-percent part of suspended sediment samples. The model was verified with hydrologic estimates and field data. Tests were directed toward keeping coarse sediment out of the headworks, making sluicing of sediment less troublesome to downriver water users, reducing trash problems on the headworks rack, and keeping sediment from interfering with downstream water measurement. A three-part sequential trap system in front of the headworks was developed. This system uses sluicing over successive trap walls and reduces the amount of sediment entering the tunnel, on the average, to about 19 percent of that entering the existing system during 1971 to 1975. At lower riverflows, the traps reduced the grain size of the sediment entering the tunnel to one-twentieth that of the riverbed material, and at higher riverflows the reduction was to one-fifth that of the riverbed material.

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by

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June 1981

Hydraulics Branch
Division of Research
Engineering and Research Center
Denver, Colorado

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R. L. Sund, K. F. Franc, and K. D. Mahnken did much of the model operation and analysis during the verification and background data collection phase of the studies. P. Julius of the Hydraulics Branch carried out most of the model operation during the modification study phases of the testing. W. M. Batts took the documentary photographs used in this report. Laboratory shop personnel were especially helpful in building the model and making many difficult alterations as the studies progressed. Final editing and preparation of the manuscript for printing were done by R. D. Mohrbacher, Technical Publications Branch.

In May of 1981, the Secretary of the Interior approved changing the Water and Power Resources Service back to its former name, the Bureau of Reclamation.

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PURPOSE AND APPLICATION

These model studies were made to help designers formulate and verify design concepts for modifying Blanco Diversion Dam. The studies were directed towards reducing sediment passing through the tunnel, making removal of debris from the trashrack easier, making disposal of sediment down river by sluicing less troublesome, and keeping the downstream V-notch weir unobstructed by sediment.

The results and experience of these studies can be used by designers to finalize their design for the proposed modification to the existing system. Besides being applicable to modification, the experience gained may be of help in designing other high mountain stream diversions with similar stream and sediment characteristics.

INTRODUCTION

Existing Diversion System

Blanco Diversion Dam is the uppermost and largest of three dams on the San Juan-Chama Project. By means of it, surplus water is transported from streams in south-central Colorado, near Chromo, for use in areas in north-central New Mexico, near Chama (fig. 1). Water from the Blanco Diversion Tunnel is combined with water manifolded from the Little Oso Diversion Dam and, further downstream, from Oso Diversion Dam.

A 15.24-m (50-ft) ogee spillway (fig. 2) with a 3.96-m (13-ft) rise from the reservoir apron was provided to pass 70.5 m³/s (2490 ft³/s) to the river downstream.

The tunnel headworks sill is set at 1.45 meters (4.75 ft) above the reservoir apron (fig. 3). The headworks was designed to pass 14.7 m³/s (520 ft³/s) through the trashrack, which has a 152-mm (6-in) bar spacing. Large flows are controlled by a 4877- by 2134-mm (16- by 7-ft) top seal radial gate (fig. 3). Smaller flows up to 1.42 m³/s (50 ft³/s) were intended to be passed around the radial gate with a 914- by 914-mm (3- by 3-ft) slide gate.

Upstream from the radial gate, a 75-mm (3-in) slot across the whole flow section floor drops sediment into a 750-mm (30-in) diameter pipe. The sediment and the required bypass flow return to the river by way of the sluiceway chute, entering the chute just downstream from the sluiceway gate (figs. 2 and 3). Just before flow enters the 2616-mm (8-ft 7-in), concrete-lined tunnel, it is measured with a 3658-mm (12-ft) Parshall flume.

The sluiceway is located at the left end of the spillway, adjacent to the right side of the tunnel headworks. Flow through the sluiceway is controlled by a 1524- by 5182-mm (5- by 17-ft) radial gate (fig. 3).

According to the Designers Operating Criteria, Blanco, Oso, and Azotea Tunnels and Diversion Facilities, 1972, all gates except the sluice gate were to be operated by automatic control. This system uses set-point and water-level stabilizing controllers. The reservoir water surface is kept at a constant elevation until the selected tunnel flow is reached; then control of the reservoir elevation is stopped and reservoir level is allowed to rise, permitting excess flow to pass over the ogee spillway. After the selected tunnel flow is reached, a sensor at the Parshall flume monitors measuring head for control of the headworks gates. The system was designed to switch from the low-flow slide gate at 1.42 m³/s (50 ft³/s) to the high-flow radial gate and vice versa.

Past and Present Performance and Operational Difficulties

During a heavy runoff of water in 1973, operators reported large quantities of sediment passing through the headworks and tunnel. The monthly hydrograph of this difficult operational year is shown on figure 4. The solid line is the riverflow, the short-dash line is the flow diverted through the tunnel, and the long-dash curve is the flow passed downriver.

Submerged trash plugged the rack and entrance to the sediment dropslot upstream from the headworks gate. The dropslot and return pipe were plugged with sediment.

Sediment bars traveled through the reservoir area, and deposits up to 1.4 m (4.5 ft) deep formed on the headworks sill. At times, sediment bars traveled to the Parshall flume. Figure 5 shows a photograph of sediment obtained at the flume by project personnel. The largest cobble in the photograph has a minor diameter of about 70 mm (2.75 in); however, field personnel report

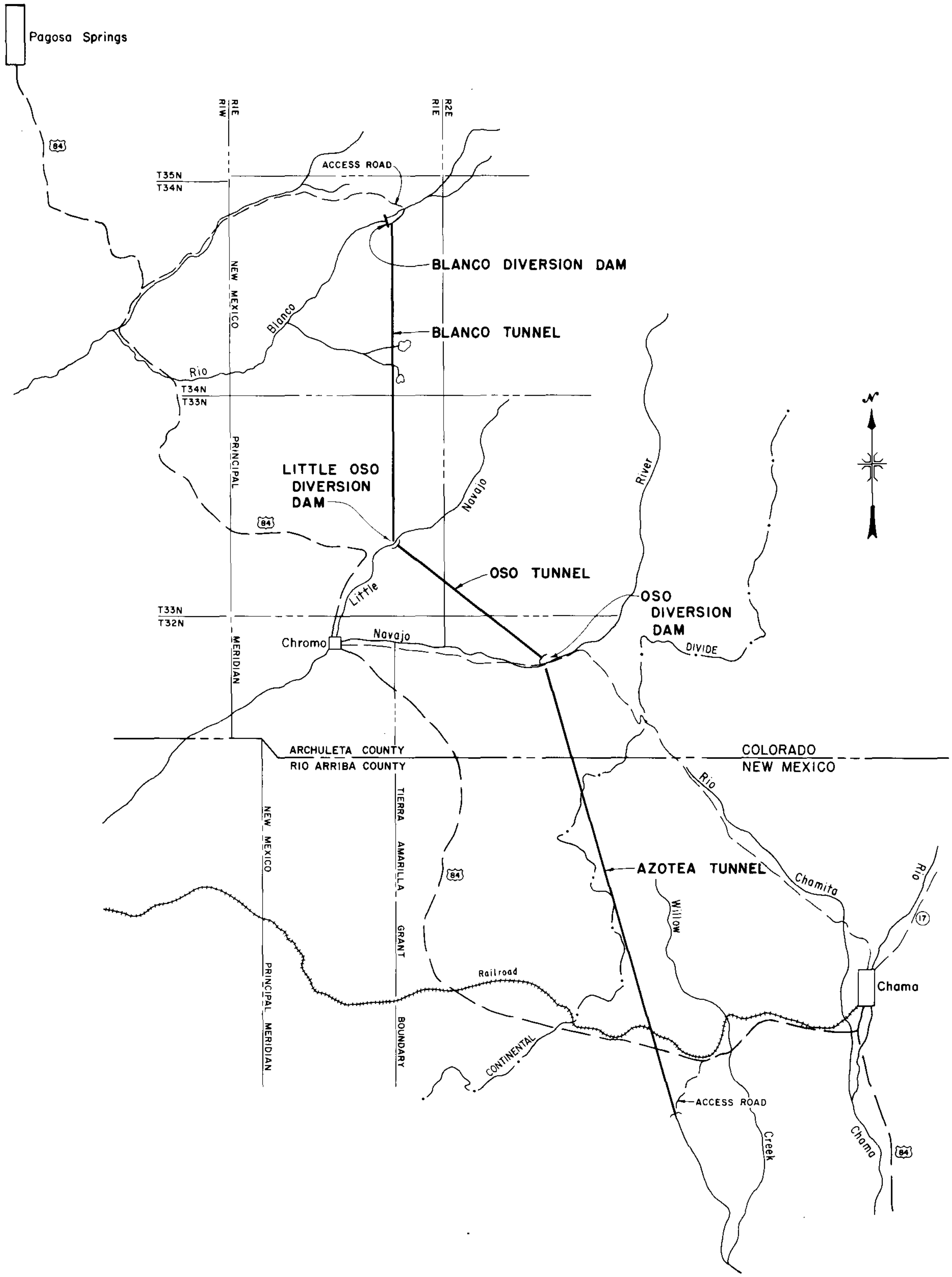


Figure 1.–Location map.

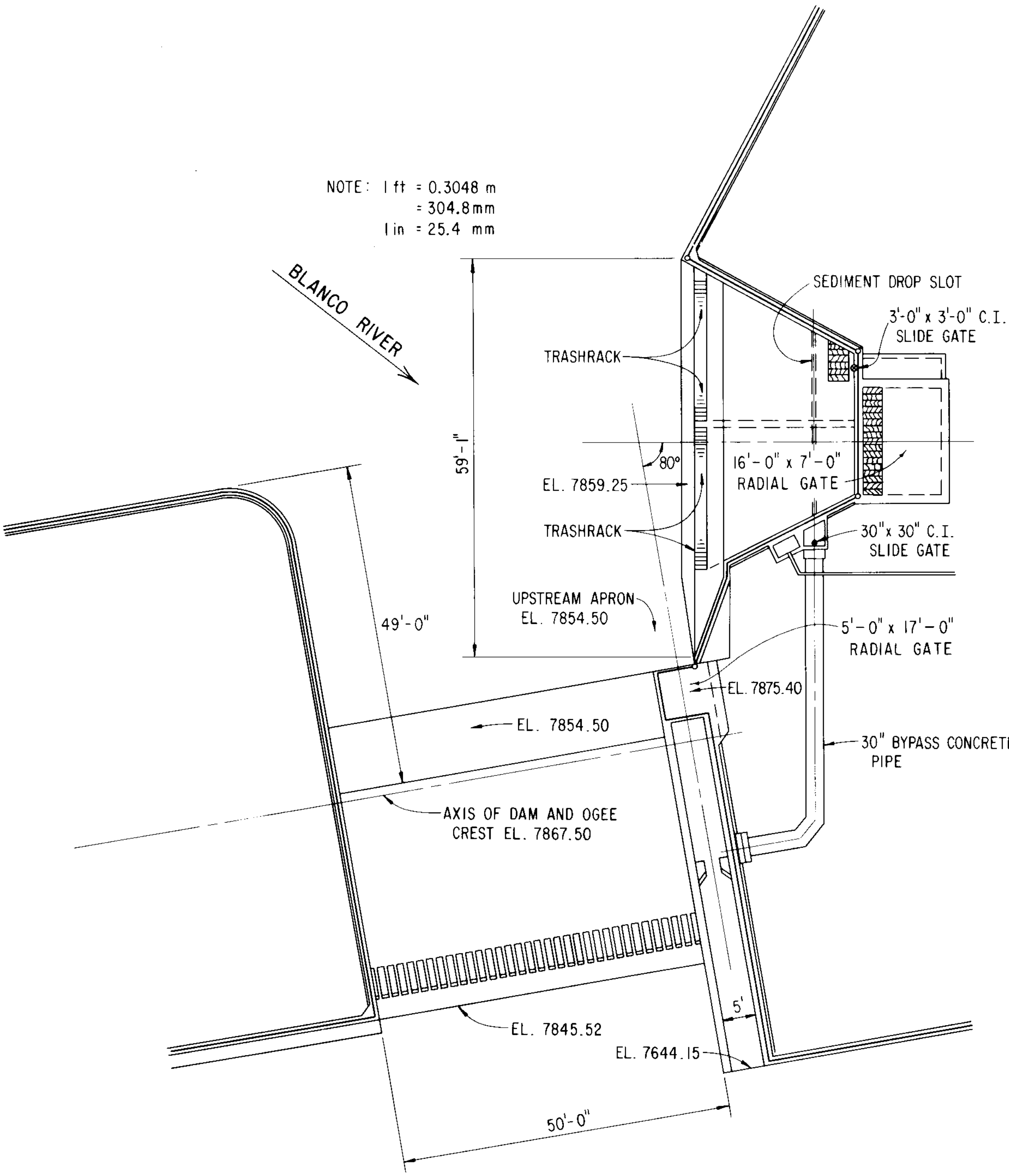


Figure 2.–General plan of existing diversion system.

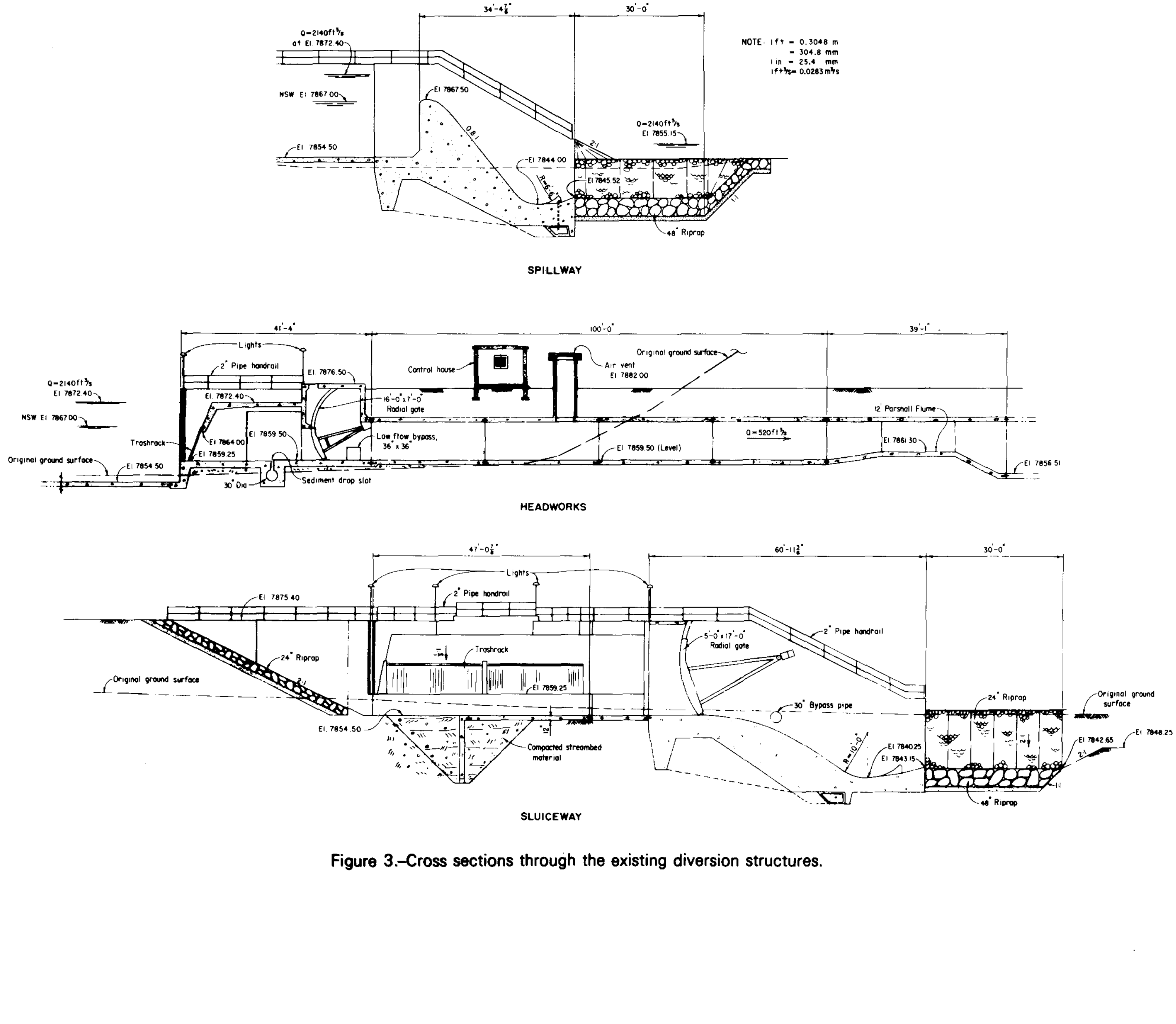


Figure 3.–Cross sections through the existing diversion structures.

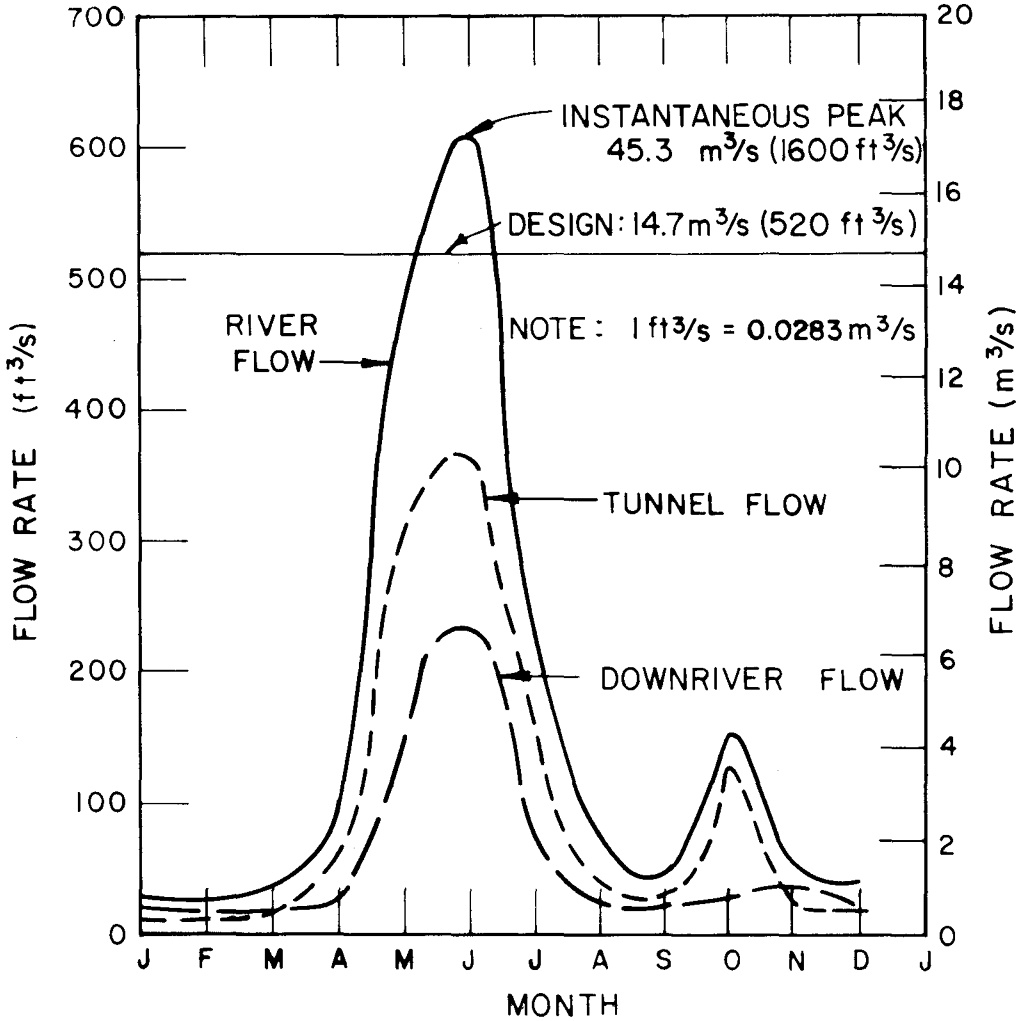


Figure 4.–Monthly hydrograph for 1973.

that the trashrack has passed cobbles up to 150 mm (6 in) in diameter.

Sediment passing through the Blanco Tunnel headworks travels through about 40 km (25 mi) of tunnel to the Azotea baffled drop structure. Examples of sediment obtained at the drop structure are shown in figure 6. The gravel tends to abrade towards ball or wheel shape as it travels through the tunnel. Operators reported the rock of this sample is uniquely identifiable as coming from the Blanco diversion rather than from the other two diversions feeding the Azotea Tunnel. A hydrologic study of 5 years of operation estimated that 68 percent of the river sediment went through the tunnel while delivering 62 percent of the river water.

After the 5 years of operation, the concrete tunnel invert has been abraded 20 mm (3/4 in) deep on about a 0.6-m (2-ft) transverse portion of the invert. However, at some slip-form construction joints, there are 150-mm (6-in) deep holes. The concrete baffled drop structure was damaged and has been repaired.

Fine, submerged debris (fig. 7) causes more problems than the larger floating log and branch type. After the tunnel discharge reaches 8.5 m^3/s (300 ft^3/s), the submerged debris starts to mat on the trashrack and restricts the discharge through the tunnel. Besides plugging the trashrack, which has interfered with automatic control of the discharge, the matting debris has plugged the entrance to the sediment

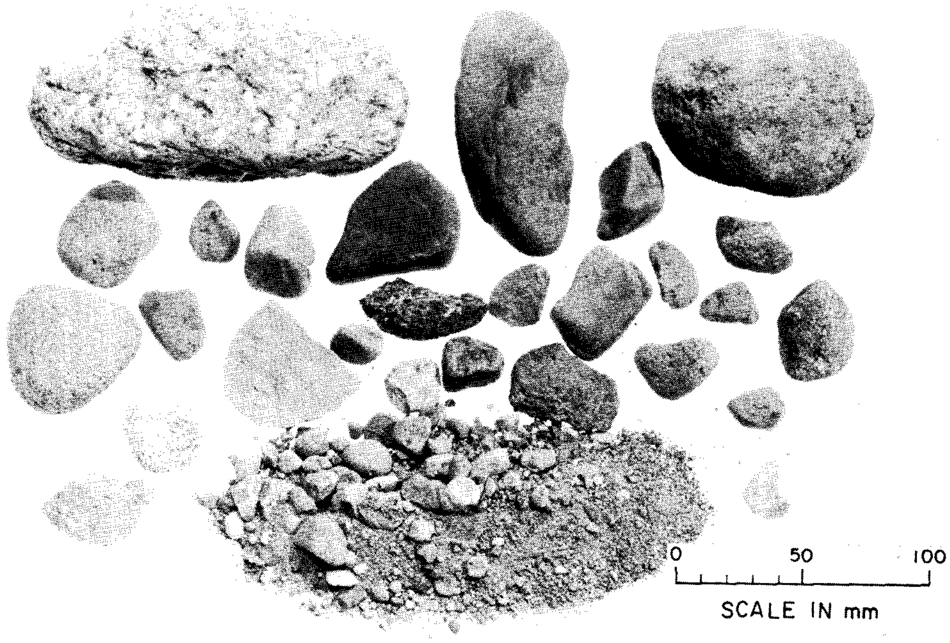


Figure 5.–Sediment sample from the Parshall flume. P801-D-79615

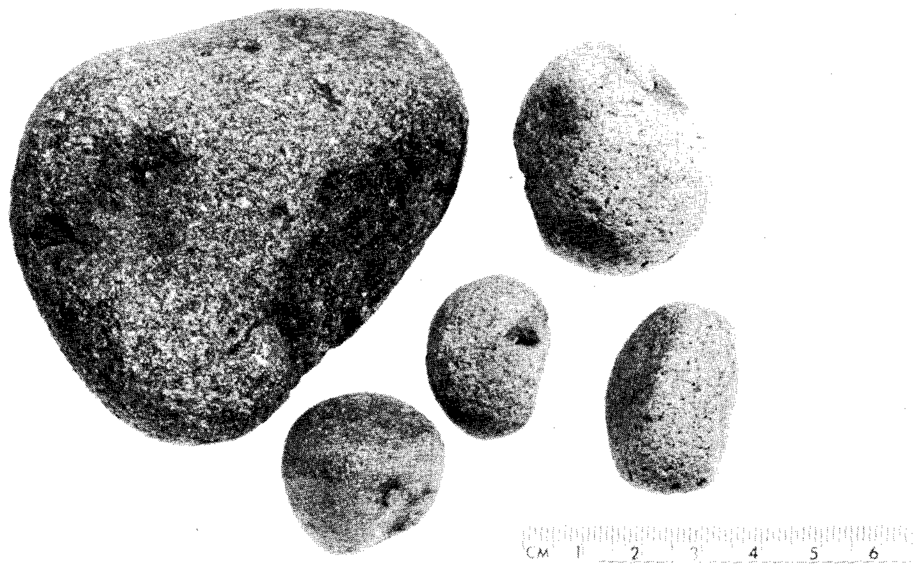


Figure 6.–Typical abraded gravel taken from the Azotea drop structure after traveling through 40 km (25 mi) of concrete-lined tunnel. P801-D-79616

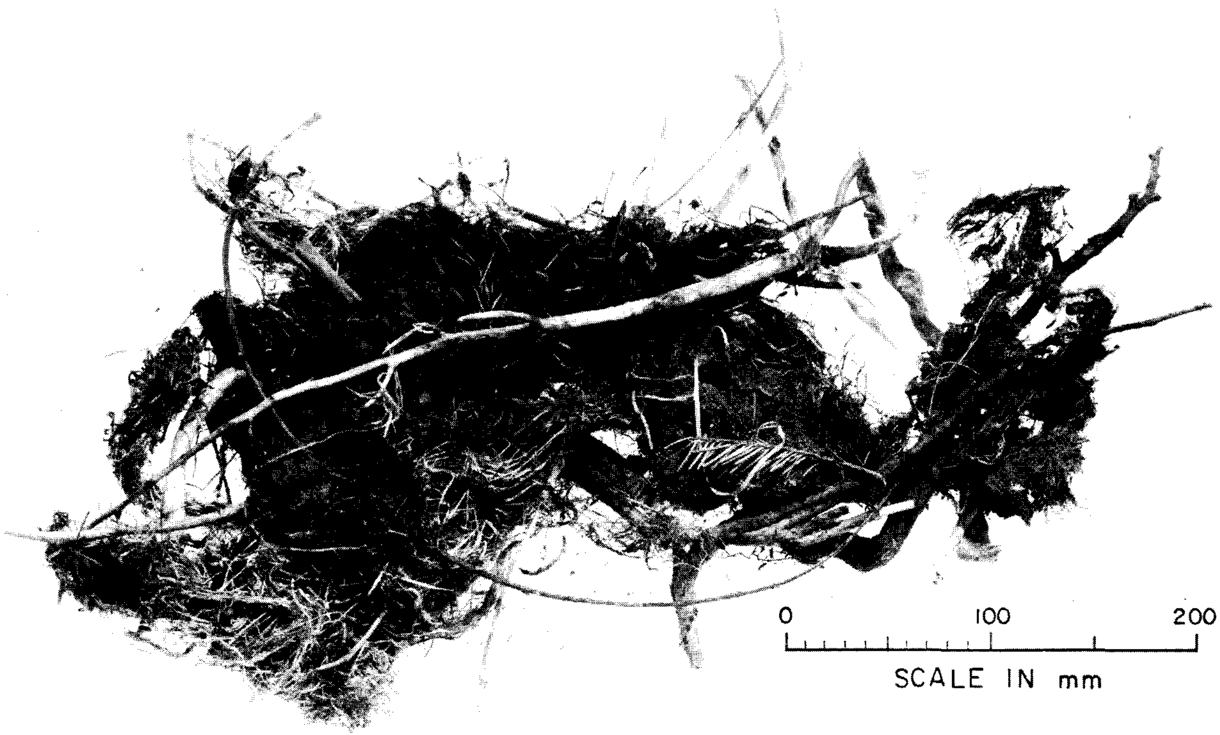


Figure 7.–Fine, submerged debris raked from trashrack. P801-D-79617

dropslot. The dropslot then cannot be cleaned and becomes ineffective in preventing sediment from entering the tunnel.

Operators report that 18 to 24 hours are required for cut-bank sluicing from in front of the tunnel intake. The optimum discharge for the sluicing is about 8.5 m^3/s (300 ft^3/s). When sluicing is attempted at larger discharges, flat-bed flow develops and the bedload transports at higher riverbed elevations during reservoir cleaning; therefore, sluicing action is considerably less effective.

Operators tried unsuccessfully to continuously move sediment through the sluiceway. Continuous sluicing tends to bring more sediment and trash closer to and in front of the headworks, with more turbulence, than when excess flow is passed over the spillway.

The V-notch weir box in the sluiceway has frequently been filled with sediment during sluicing, interfering with measurement of bypass water. Also, there have been complaints that sluice sediment was interfering with downstream irrigation.

Because of the above problems, the Hydraulics Branch was requested to perform a hydraulic sediment model study of the diversion system to help finalize design concepts for modification of the structure.

General Objectives and Work Plan

The direction of these studies, besides being determined by the numerous operational difficulties, was further complicated by several years of litigation. Nevertheless, during several meetings with regional personnel and designers, priorities were set by consensus in the following order:

(1) Keep coarse gravel out of the tunnel headworks,

(2) Make sluicing less troublesome at downstream irrigation turnouts,

(3) Reduce trash problems on the headworks trashrack, and

(4) Keep sediment out of the V-notch weir box or recommend some other measuring device.

It should be noted that the relief of some of these problems adds to the distress of one or more of the others. Therefore, optimizing modification within these four problem areas was necessary.

To solve the problems, laws of similitude were investigated first to enable the basic model to be designed and constructed. Then, measuring and operating techniques for the model were developed. A search for or generation of hydrologic data was done and simulated hydrographs and sluicing runs were made with the existing diversion arrangement to verify the model with available prototype data. Then, various modifications were tested and the results compared to determine the best scheme for the modification at Blanco Diversion Dam.

CONCLUSIONS

Keeping sediment out of the tunnel is of the highest priority; therefore, the trap system shown on figures 33 and 34 is recommended for the modification design. This system permits operation between two bounds (fig. 35) on the amount of river sediment delivered to the tunnel. One is with the traps continually kept clean by intermittent sluicing and the other is by allowing the traps to remain continually full of sediment.

The average delivery through the tunnel for the 1971-75 period was 62 percent of the riverflow. Both the model and a hydrologic study indicated that about 68 percent of the moving sediment in the river went into the tunnel with the existing diversion system. Using these values as a comparison base, the capability of the recommended modification can be estimated as follows:

1. With traps functioning, sediment intake by the tunnel will be about 19 percent of that for the existing diversion system. This should help considerably in reducing tunnel invert abrasion.

2. With traps left uncleaned or unattended, the tunnel sediment intake will be about 44 percent of the base-period quantity for the existing diversion system arrangement. The abrasion reduction will naturally be less than in (1) above.

3. For traps kept clean and at a riverflow of 14.2 m^3/s (500 ft^3/s), the 50-percent size of the sediment going into the tunnel will be about one-twentieth that of the bed material. At 31.7 m^3/s (1 120 ft^3/s), the sediment size reduction will be about one-fifth. These reductions should reduce tunnel abrasion.

4. With traps kept clean by overwall sluicing, about 2-1/2 times as much sediment will go downriver as for the base time period with the existing diversion system. The Region should continue consideration of the idea of providing sluiceable turnouts for downriver irrigators affected by the sediment.

5. With traps left full, about twice as much sediment as for the base period will go down the river. Sluiceable turnouts may still be advisable under this condition. More sediment goes downriver than with the existing system because the outermost trap wall guides excess flow under narrow gates further away from the headworks.

6. With the traps kept clean and at a riverflow of 14.2 m^3/s (500 ft^3/s), 43 percent of the river sediment goes through the radial gates outside the trap. At twice this flow rate, 68 percent of the sediment passes through the gates outside of the traps.

7. For a hydrograph similar to the 1973 flood, the traps would have to be sluiced twice during the flood to minimize the tunnel sediment intake. For total cleaning, about 258 m^3 (9 110 ft^3) of sediment would be sluiced out of the traps. Each sluicing would take from 25 to 60 minutes for total cleaning, including gate travel times. Total reservoir cut-bank sluicing now taking about 24 hours would no longer be necessary.

8. In time, operators may find it to be expedient to partially clean the traps or,

anticipating large riverflows and sediment loads, to clean early.

Other advantages of incorporating the trap arrangement into the modification design are:

1. Floating trash tends to back away from the trashrack when sluicing the inner trap.

2. Sluicing can be done directly in front of the headworks via traps without moving major amounts of the sediment from reservoir bed deposits.

3. Trap walls permit sluicing of the traps without closing the headworks gate.

4. If desirable or necessary, relatively clear waterflow can be continued after cleaning the traps to dilute downriver sediment.

5. The traps and the spillway gates make possible the dewatering of the trashrack and the sediment dropslot and return pipe in the floor of the tunnel entrance even during high riverflows. Dewatering would make maintenance easier and this capability would make the entrance an ideal place to test and develop automatic or motorized trashrack cleaners.

Of other possible solutions tested, continuous sluicing with three narrow gates and without traps proved to be of some help, but only for deliveries less than 45 percent. Using base water delivery of 62 percent, continuous sluicing would result in about 1-1/3 times the sediment intake to the tunnel than for existing system.

THE MODEL

Model Similitude

The dynamic variables for the model were scaled by the Froude Law; thus, for any selected geometric scale, the ratios for flow rate, velocity, and time are:

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

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

$$T_r = L_r^{1/2} \quad (3)$$

where:

L_r is the length ratio, prototype-to-model.

Blanco River is a high mountain stream with bed sediment ranging from cobbles to fine sand; therefore, segregation and armoring could be important in regard to the river sedimentation process. So, it was decided to represent as much of the sediment particle size distribution as possible using settling velocity to scale the sediment particle sizes.

A geometric model scale of 1:16 was selected; thus, from equation (2) the velocity scale ratio is 1:4. The gradation for pit-run sand frequently used in the laboratory was scaled to the prototype on the basis of settling velocity. This gradation was compared with gradations of a bed and a suspended sediment sample collected by the Sedimentation Section, a sample of the 1973 flood sluicings collected by the author, and a 1975 sample from the Parshall flume collected by project personnel (fig. 8). The scaled pit-run sand gradation is within the range of the field gradations and was therefore satisfactory for the model investigation.

For flow boundary surface-resistance consideration, the larger sizes down to 73 percent of the finer sizes of the sediment scaled by settling velocity agreed to within 15 percent or better with diameters scaled by the geometric scale ratio.

The governing equation used for friction and slope for the model was:

$$S = f\,\frac{V^2}{8gR} \quad (4)$$

where:

S = Slope

V = Velocity

g = Acceleration due to gravity

R = Hydraulic radius

$f = \phi\left(\text{Reynolds number}, \frac{K}{4R}\right)$

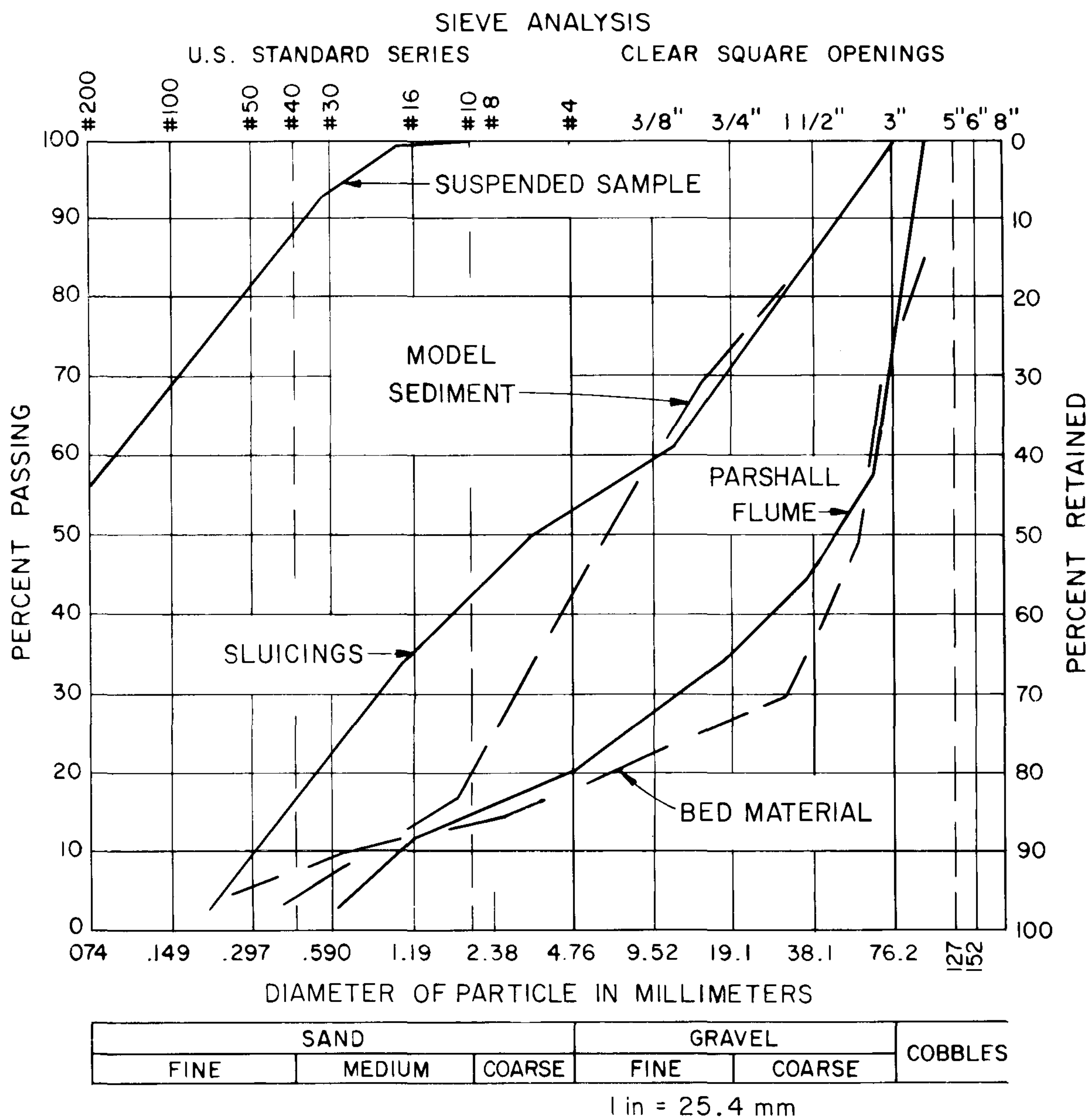


Figure 8.–Comparison of model and prototype sediment grain-size distributions.

where:

K is the boundary surface roughness.

In equation (4), $V^2/8gR$ can be grouped. This is proportional to the Froude number squared and the ratio of model to prototype of this group is equal to 1. Thus,

$$\frac{S_p}{S_m}=\frac{f_p}{f_m}$$

For wholly rough flow, f is a function of $K/4R$ only. Because R scales geometrically and K is a function of larger particle sizes that scale geometrically, then:

$$\frac{S_p}{S_m}=\frac{f_p}{f_m}=1 \qquad (5)$$

Equations (1) through (5) are the theoretical scaling relationships; however, when modeling includes sediment, distortions occur almost without exception. The nature of the distortions must be found and then eliminated or corrected by adequate comparisons of model to prototype performance as discussed later in this report.

General Description of the Model

The model was built to a geometric scale of 1:16, covering the area shown on figure 9. The model was split into two basic levels; the higher level represented about an 83.8-m (275-ft) long approach to the dam from the reservoir, and the lower level represented the area from the stilling basin to a downstream basin dam, including a 120° V-notch weir for measuring bypass flow. Figure 10 shows the sediment pump that pumped sand and water, representing riverflows up to 32.0 m^3/s (1130 ft^3/s), through 75-mm (3-in) piping to the upstream end of the model. The sediment-return sump was made as small as possible to increase the model response to hydrograph flow, to changes in rate of sediment transport, and to conserve sand. However, because of this quick response, the downstream pool had to be partially cleaned during sluicing tests rather than recirculating the large amount of sediment.

An auxiliary water supply was provided by a main laboratory pump and was used with the main venturi meters to calibrate flows of the component parts of the model. This auxiliary

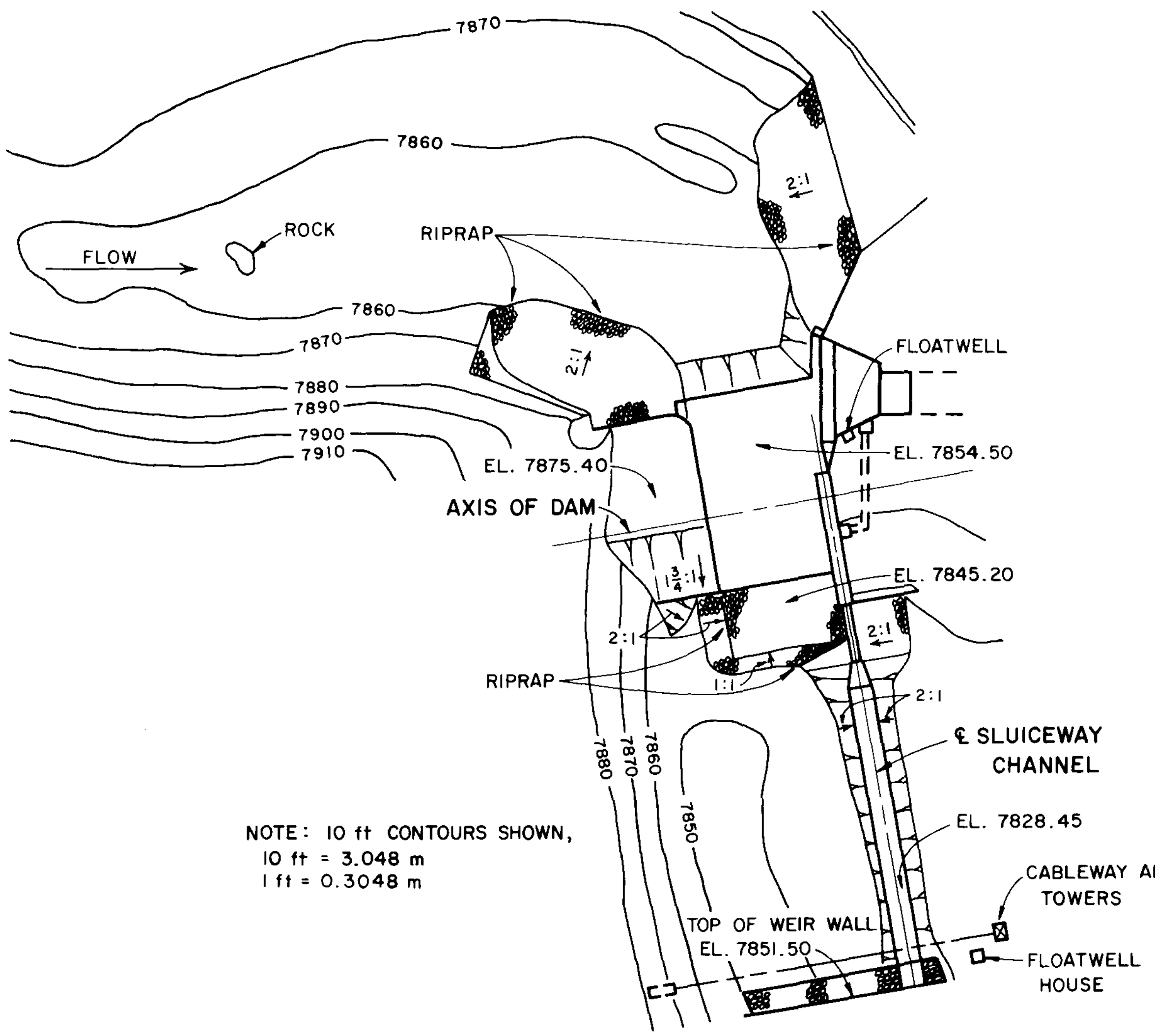


Figure 9.–General area modeled.

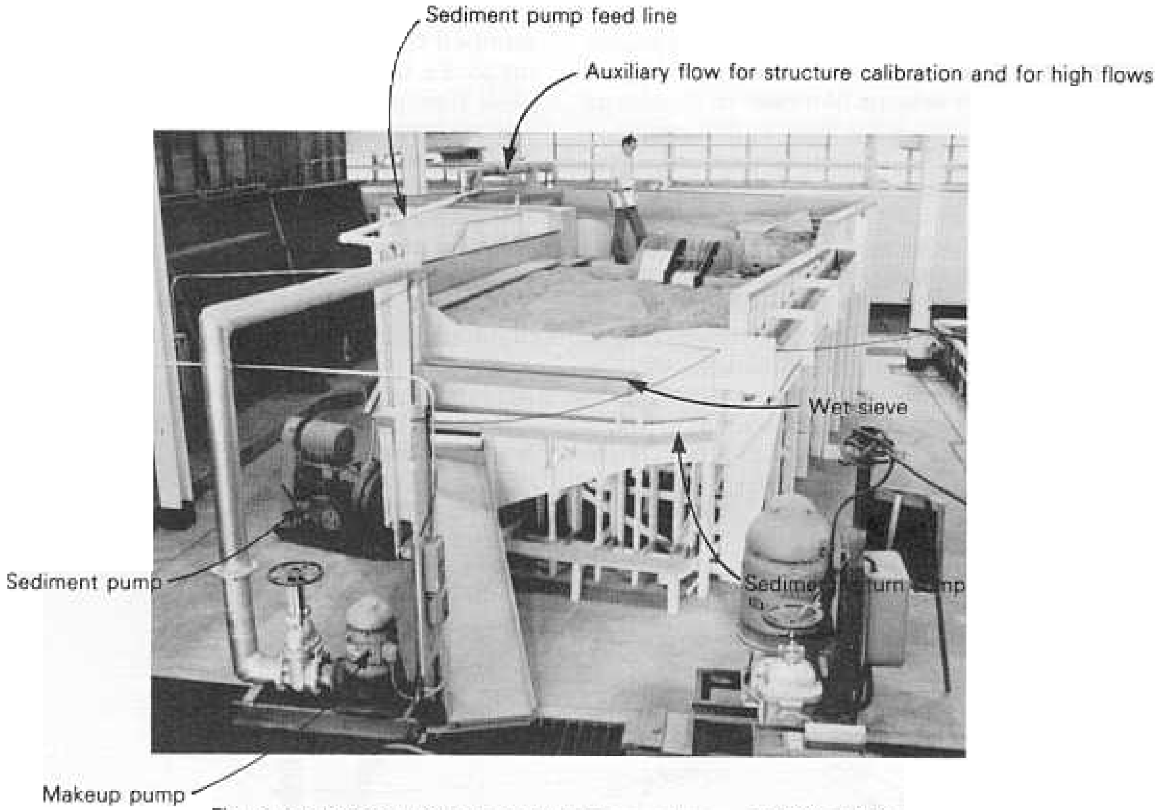


Figure 10.–General view of model looking upstream. P801-D-79618

supply was also used occasionally for required flows that exceeded the sediment pump capacity.

For operational tests, such as sluicing or changing gate settings, provision was made for handling transients caused by reservoir volume changes. When the reservoir water surface was rising, the makeup pump in the left foreground of figure 10 was continuously operated at a small discharge. When the model water elevations were constant or falling, the weir and chute in the middle foreground were used to return water to laboratory storage.

The main difficulty with the model was that the quantity of sediment transported was not automatically adjusted with respect to riverflow changes. Therefore, sediment was added or subtracted to force the model to form the bed slopes needed to transport sediment at rates estimated by the Sedimentation Section (fig. 11). When hydrographs were programed, the results of model operation were computed in the ratio of diverted component sediment loads to the river sediment load. Doing this partially compensated for not operating at the exact sediment transport rate. However, care was exercised to make sure that the deviation of the model sediment loads from the prototype was not too large. If deviations are too large in such modeling, slopes and bed forms can be altered enough to affect the quantities of sediment transported in the divided flows.

Because the model sediment was sufficiently coarse, samples were obtained by wet sieving through No. 100-mesh screens during measured time intervals. The wet sieve that was used to measure downriver sediment is shown in figure 10. By settling-velocity scaling, the wet sieve will catch 0.52-mm and larger prototype particles. This includes sizes equal to the largest 7-percent sizes of field-suspended samples.

The headworks was made of plexiglass and included the sediment dropslot with return piping to the sluiceway (fig. 12a). The trashrack was made of strips of sheet metal spaced on brass welding rods (fig. 12b). Spillway and sluiceway crests were fabricated from high-density foam plastic (fig. 13) and sluiceway

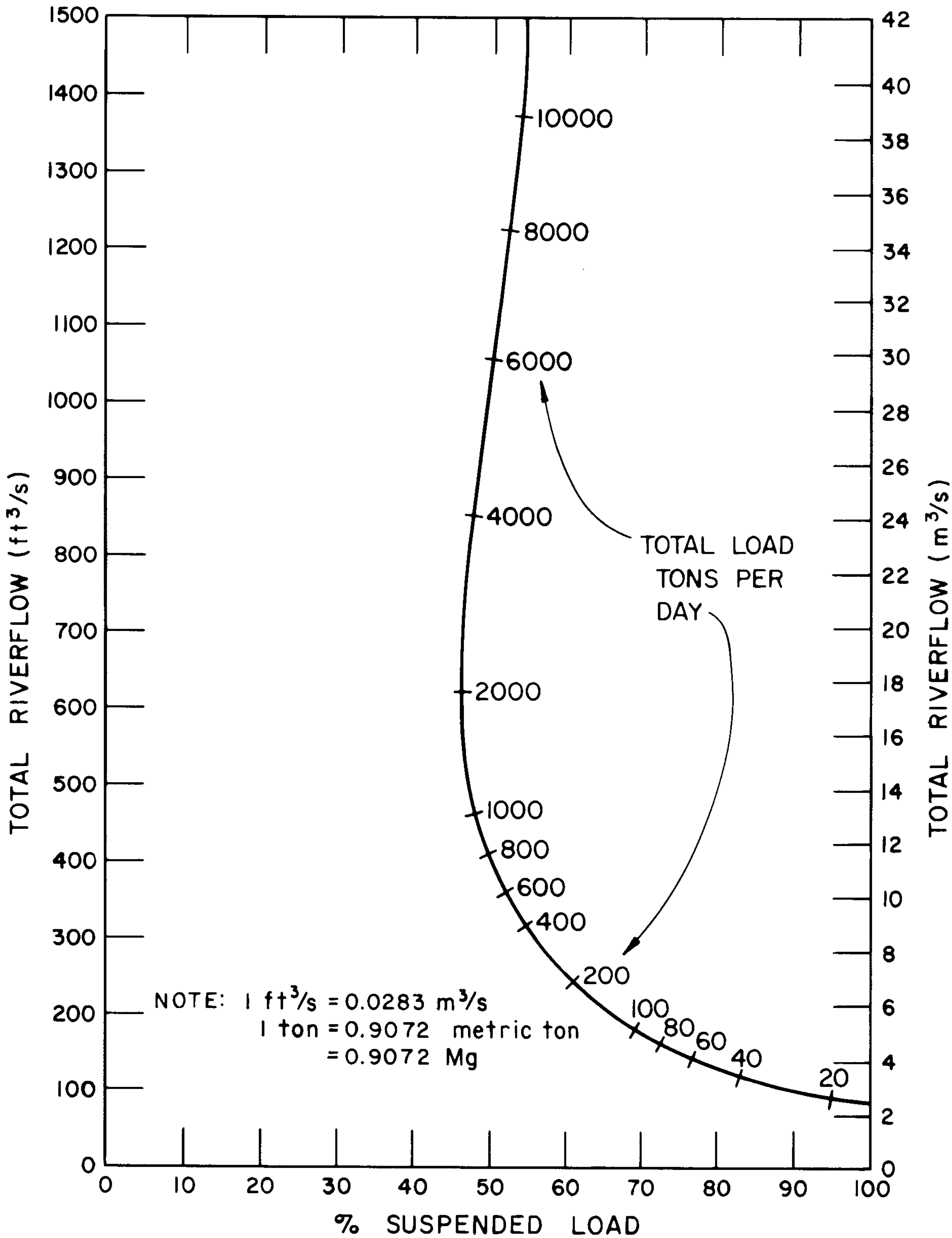
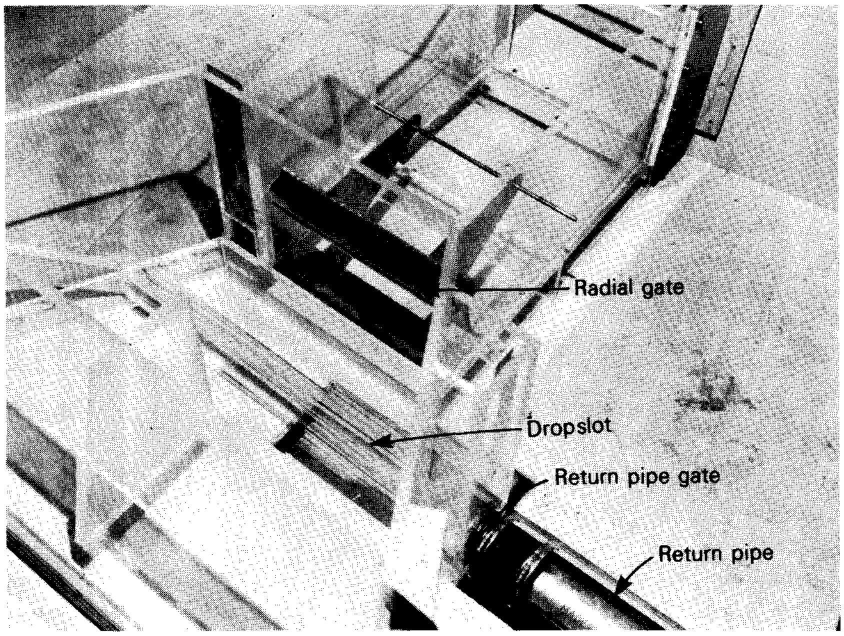
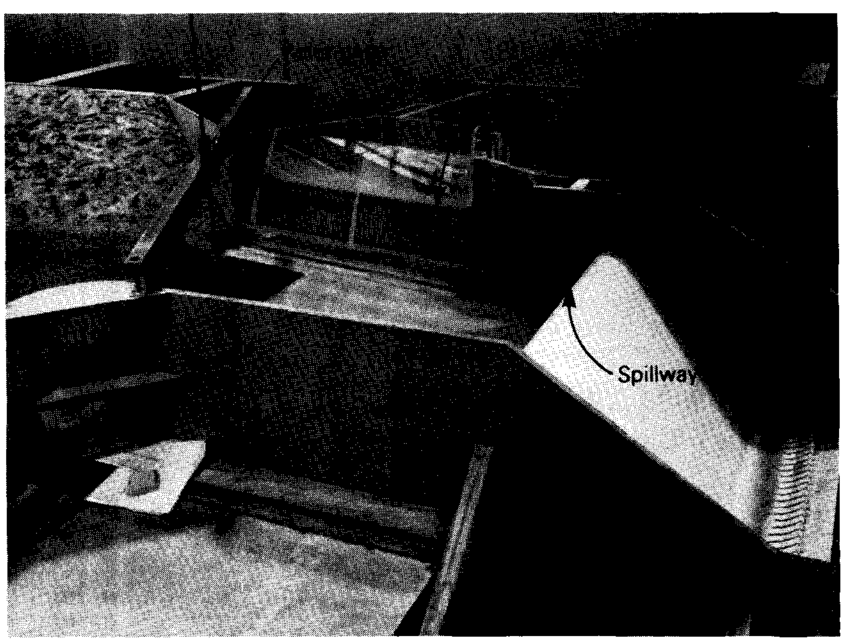


Figure 11.–Sediment discharge rating curve.



(a) Tunnel headworks showing sediment dropslot, return piping to the sluiceway, and the topseal radial gate. P801-D-79619



(b) View showing relationship of the tunnel headworks to the spillway and sluiceway. P801-D-79620

Figure 12.–Closeup views of model of existing diversion system.

and headworks radial gates (fig. 13) were fabricated from sheet metal. A vernier point gage was used to measure the heads on the spillway, sluiceway, and on the headworks (fig. 12b).

Riprap was simulated in two ways. The first representation (top left in fig. 12b) was formed of roughened mortar supported on wood framing and metal lath. This type of riprap simulation was used on the approaches to the headworks. The riprap downstream of the headworks and sluiceway was formed with gravel scaled to represent prototype rock size (fig. 14). The downstream V-notch weir is shown in figure 15.

The model prior to operation is shown in figure 16. Sand was placed by shovel and screened according to contours given on the specifications drawings for the original diversion system as shown on figure 9.

MODEL STUDIES OF THE EXISTING DIVERSION SYSTEM

Bed Conditioning Runs

The model was run continuously at 32.0 m^3/s (1130 ft^3/s), with 14.2 m^3/s (500 ft^3/s) flowing through the tunnel headworks, 16.7 m^3/s (590 ft^3/s) passing over the spillway, and 1.13 m^3/s (40 ft^3/s) through the sluiceway. Sand was continually added during operation to build up the bed and a sufficient slope to deliver the proper amount of sediment by pump recirculation alone. Except for the very fine part of the sediment grain distribution, the sediment traveled as a delta-front sediment bar toward the reservoir area (fig. 17). Figure 18 shows the same delta-front bar after it reached the spillway, sluiceway, and headworks. The sediment moved downstream from the reservoir by traveling over the right side of the spillway first. Next, the bar went into the headworks, then the toe of the bar reached the sediment dropslot and sluiceway entrance area. After passing over the spillway and through the sluiceway, a delta-front bar developed in the downstream weir basin (fig. 19) and continued on toward the 120° V-notch weir. Plugging of the sediment dropslot occurred in two different ways:

1. The particles of sediment larger than the slot width were caught by and bridged across the slot, and as the particles accumulated, the discharge and velocity continually decreased in the return tube. Finer sediment passed between the trapped particles and the return piping eventually became fully packed with sand because the diminished flow could not flush the pipe. This was generally the way the slot would plug when the delta-front sediment bar moved across the entire headworks parallel to the slot. In the prototype, this process would be accelerated by submerged organic debris matting over the slot.

2. A point of sediment would approach the slot, dropping sediment into the return pipe, where it would pile up at the submerged angle of repose at the bottom of the 750-mm (30-in) pipe. The part of the return pipe on the side away from the gate would fill rapidly because there was no longer downstream waterflow on that side. The repose slope would then migrate toward the gate until the pipe was fully packed.

With either type of plugging, it was impossible to clean the slot and pipe when delivering water through the tunnel. Cleaning could only be done manually during periods of riverflow below about 4.25 m^3/s (150 ft^3/s) and using the sluiceway to keep the water level below the headworks sill.

Hydrograph Runs

Seven repetitions of a simulated spring runoff hydrograph were discharged through the model. Each hydrograph was programed through the model in target steps from 8.5 m^3/s (300 ft^3/s) to 19.8 m^3/s (700 ft^3/s), to 31.1 m^3/s (1100 ft^3/s), and then back to 31.1 m^3/s and 19.8 m^3/s. The flood hydrograph was estimated from the author's own observation of a spring runoff and from operators' reports and records. During the seventh hydrograph, sediment samples were obtained for the tunnel, spillway, and sluiceway flows. During all the hydrograph runs, a continuous flow of 1.13 m^3/s (40 ft^3/s) was bypassed through the sluiceway rather than through the sediment dropslot because this was the way the project was operating at the time.

The seventh model hydrograph (fig. 20), lasting 10 prototype hours, shows how the riverflow was divided through the parts of the system. Maximum tunnel discharge that the operators could deliver was about 11.8 m^3/s (415 ft^3/s)

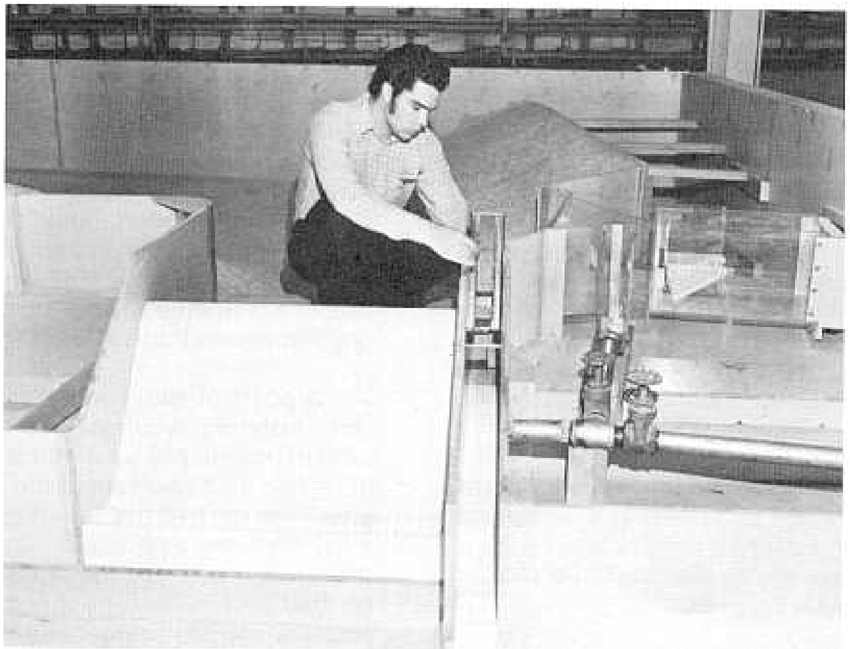


Figure 13.–View of spillway, sluiceway, slotted buckets, and sluiceway and headworks radial gates. P801-D-79621

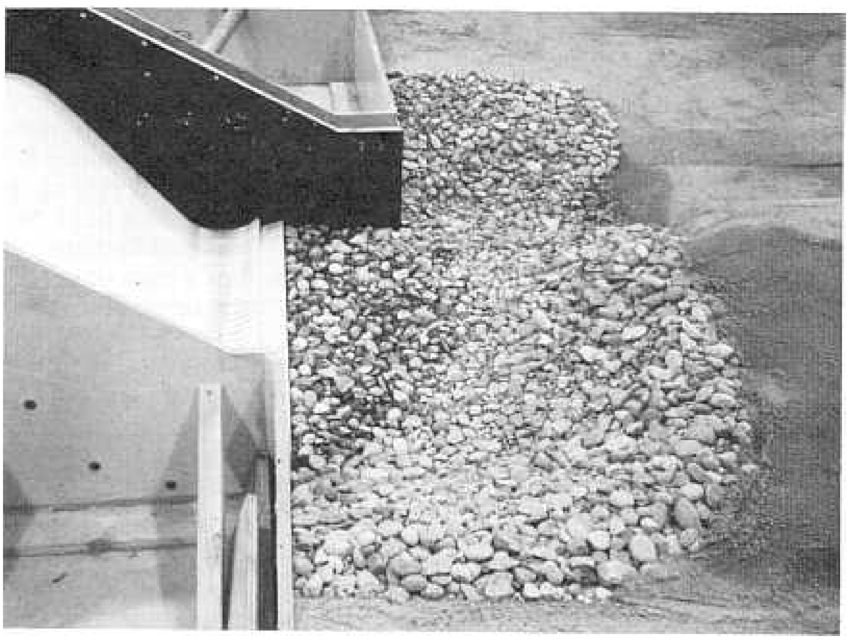


Figure 14.–Closeup of riprap modeled by gravel in the downstream basin area. P801-D-79622

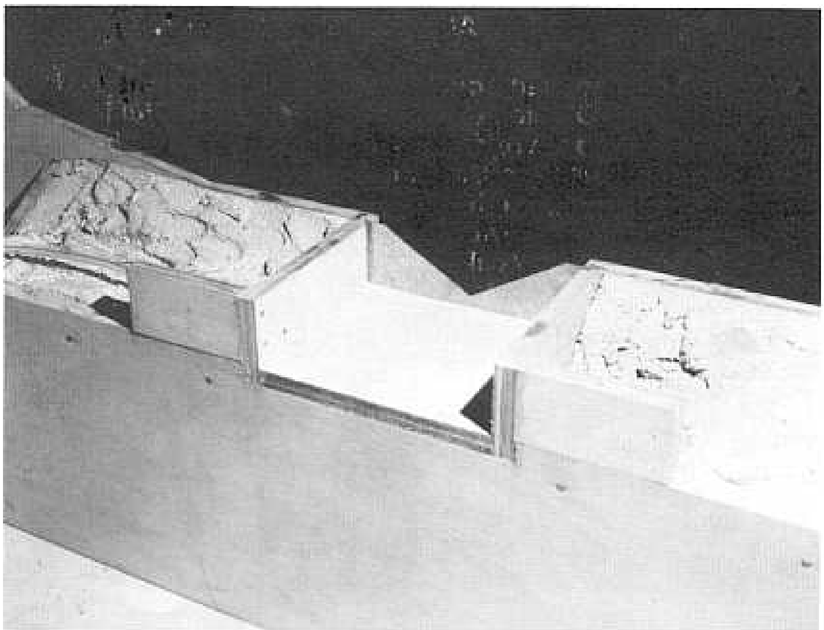


Figure 15.–View looking downstream at 120° V-notch weir before placing sand in the model. P801-D-79623

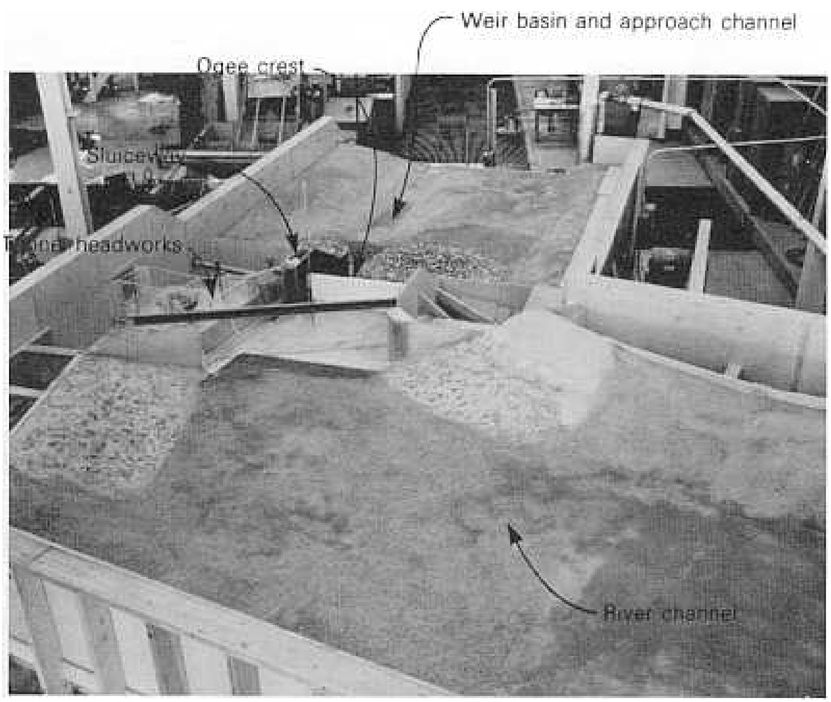


Figure 16.–General view looking downstream at the model prior to operation. P801-D-79624

Figure 17.–The delta-front sediment bar approaching the diversion structures.
P801-D-79625

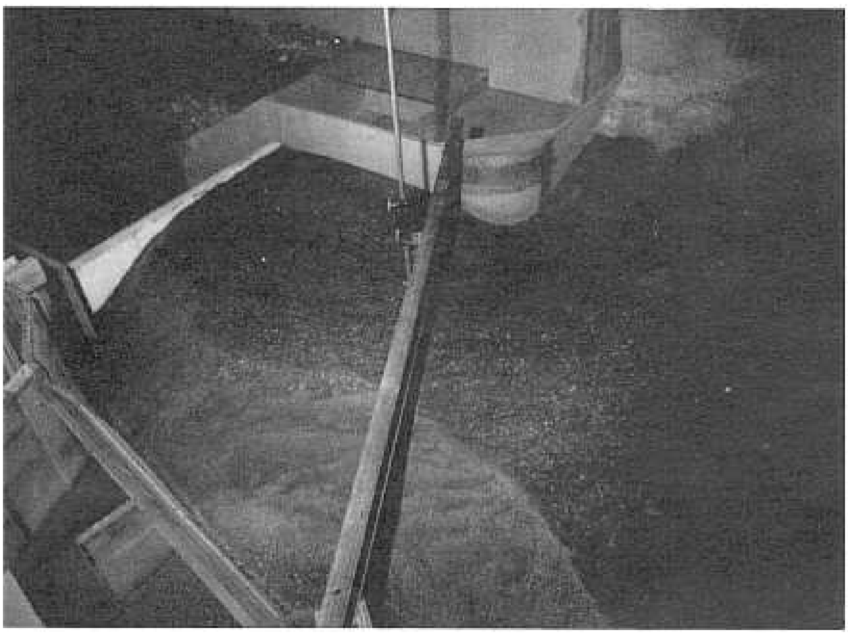


Figure 18.–The delta-front sediment bar after reaching the spillway.
P801-D-79626

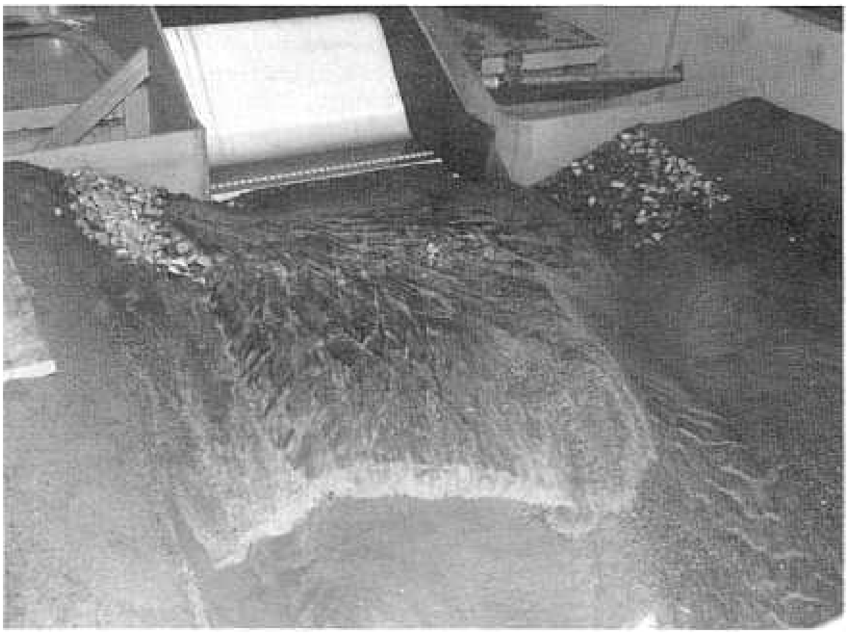


Figure 19.–The delta-front sediment bar traveling through the downstream river and weir approach channels. P801-D-79627

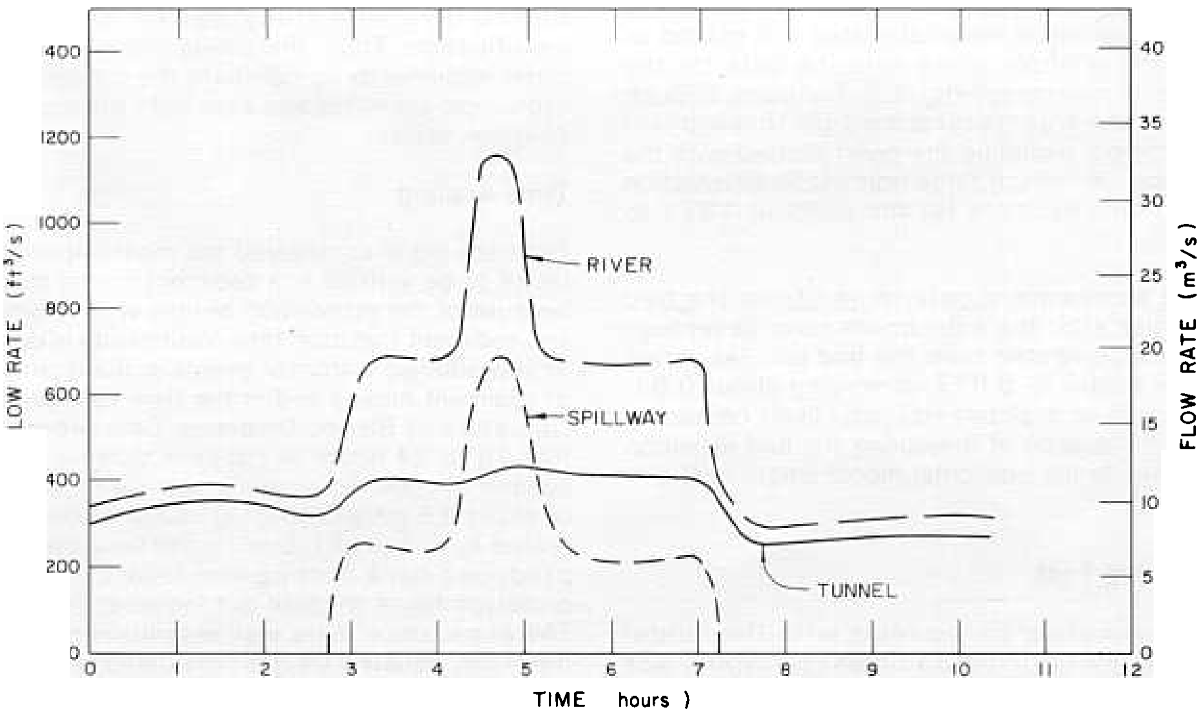


Figure 20.–Flow division during seventh hydrograph run.

because trashrack plugging caused automation difficulties in controlling the tunnel flow when the diversion was left unattended overnight. Sediment quantities are plotted on figure 21 in the form of ratios of their concentration in component flows to the total river concentration. Length of time the model was operated at the target discharges is shown by the vertical lines on figure 21.

Average flow rates from figure 20 were used to calculate tunnel headworks flow ratios, Q_H/Q_R. These ratios, along with average sediment concentration ratios, S_H/S_R, from figure 21, were used to calculate the percent of riverwater going into the tunnel headworks and the percent of sediment going into the headworks. These data are shown plotted as circle symbols on figure 22. The numbers by the data points are values of total riverflow.

Equilibrium Runs

Sand was added or extracted from the model while operating at constant riverflow and constant diversions until the proper bed slopes were attained to carry the load indicated by figure 11. Successive sediment samples were used to determine when equilibrium was reached. After equilibrium, sediment ratios and discharge ratios were calculated and plotted as square symbols along with the data for the seventh hydrograph (fig. 22). The curve through the data was determined by three-point averaging, including the point plotted with the symbol "X," which came from the Sedimentation Section's estimate for the years of 1971 to 1975.

Bed slopes were determined from the bed profiles after the equilibrium runs. Seventeen slopes computed from the bed profiles varied from 0.007 to 0.017, averaging about 0.01. This spread in slopes was most likely caused by the imprecision of measuring the bed elevation relative to the horizontal model length of 2.6 m (8.5 ft).

Sluicing Test

Preliminary sluicing tests with the model indicated optimum sediment transport was accomplished by bank cutting at discharges near 8.5 m^3/s (300 ft^3/s). Flat-bed flow developed at higher discharges, making sluicing considerably less effective. Figure 23 shows the model before, various times during, and after sluicing near optimum discharge. The reservoir area near the headworks was cleared of the sediment in about 24 prototype hours of sluicing. After sluicing, cross sections were measured to calculate quantities of sediment sluiced. The calculated amount of sediment sluiced represented about 1560 m^3 (55,000 ft^3) prototype.

MODEL VERIFICATION

General

When possible, all sediment models should be verified for model-to-prototype conformance. If a model is not similar to the prototype in geometry, motion, and force, then care must be taken in interpreting data from the model tests. If distortions are large or numerous, a model will not represent the prototype, and interpretations or data analysis will not always provide adequate correction.

Unlike the usual design model studies, the Blanco diversion study was for structure modification. Thus, there was more than the usual opportunity to compare the model with hydrologic estimates and data from the existing diversion system.

Time Scaling

Time scaling is considered the most important factor to be verified in a sediment model study because of the association of time with velocity and sediment transport rate. Verification is done by reproducing historical events in the quantity of sediment moved and in the time to move it. Operators of Blanco Diversion Dam reported that 18 to 24 hours of cut-bank sluicing were needed to clear the reservoir at a riverflow rate of about 8.5 m^3/s (300 ft^3/s). In the model at a scaled flow rate of 8.5 m^3/s, the flow velocity produced bank cutting and took about 24 prototype hours to clean out the reservoir area. Therefore, since there was essentially no time distortion, equation (3) was considered valid, and ordinary Froude scaling applied.

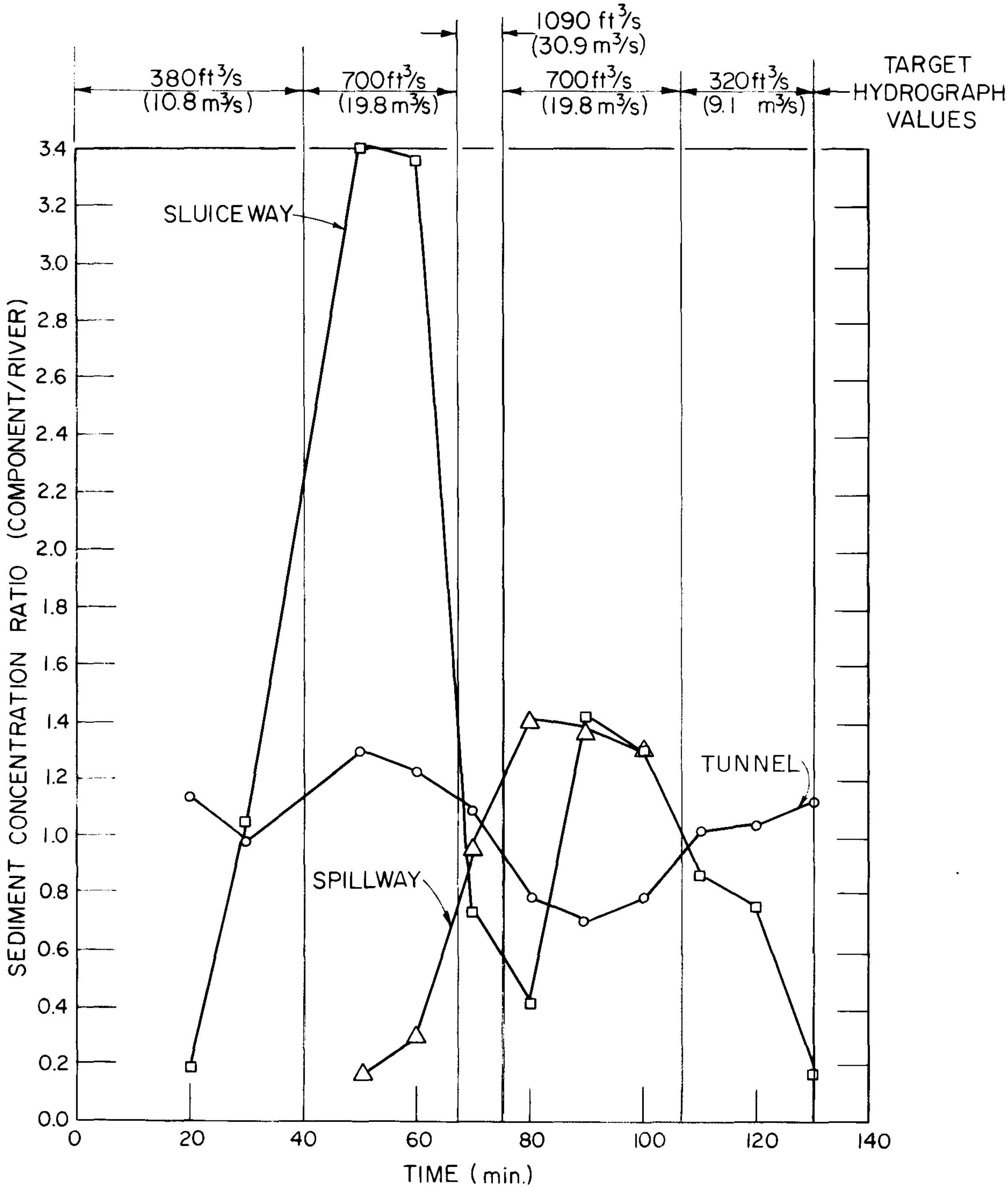


Figure 21.–Sediment concentration ratios during seventh hydrograph run.

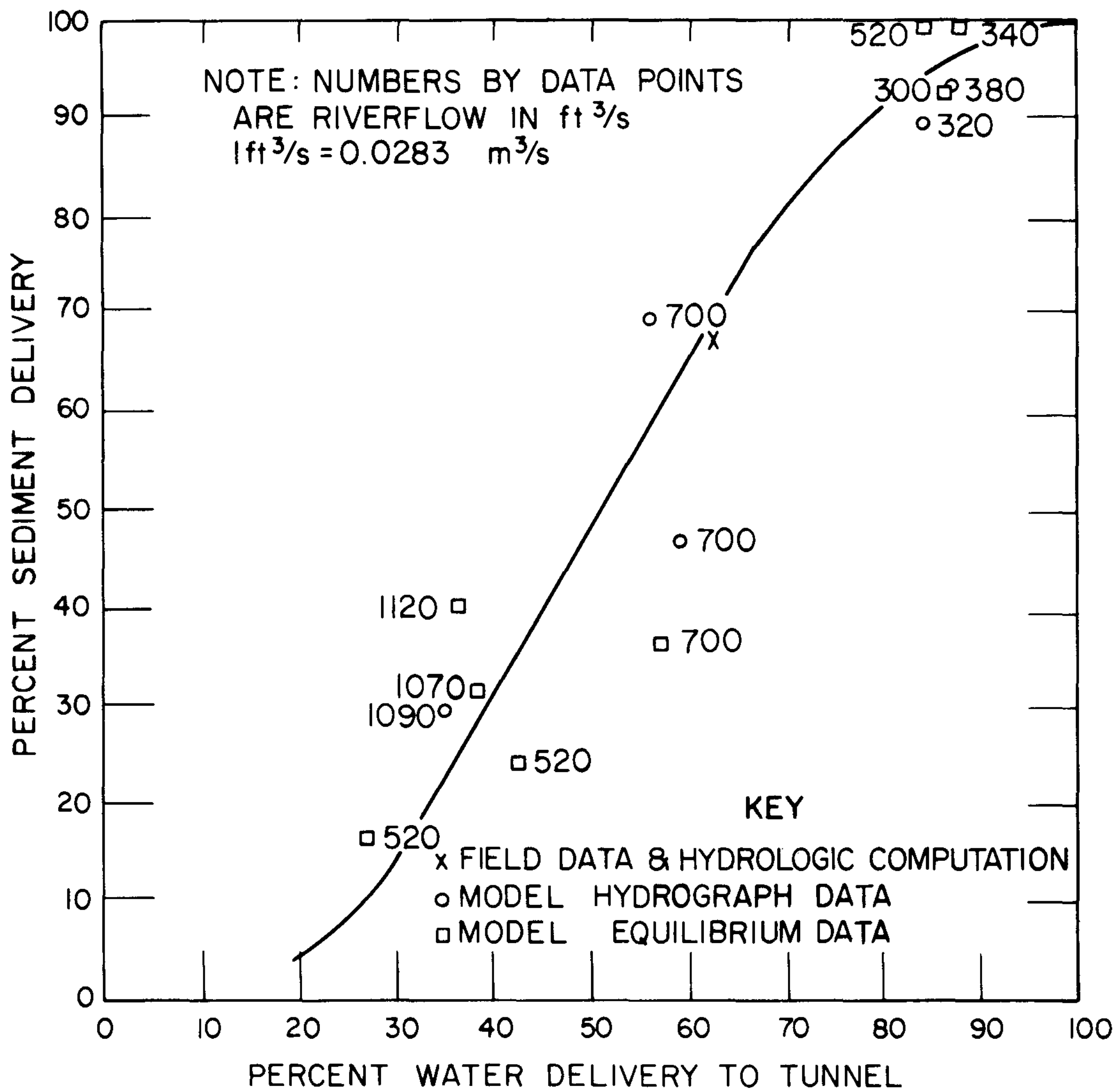
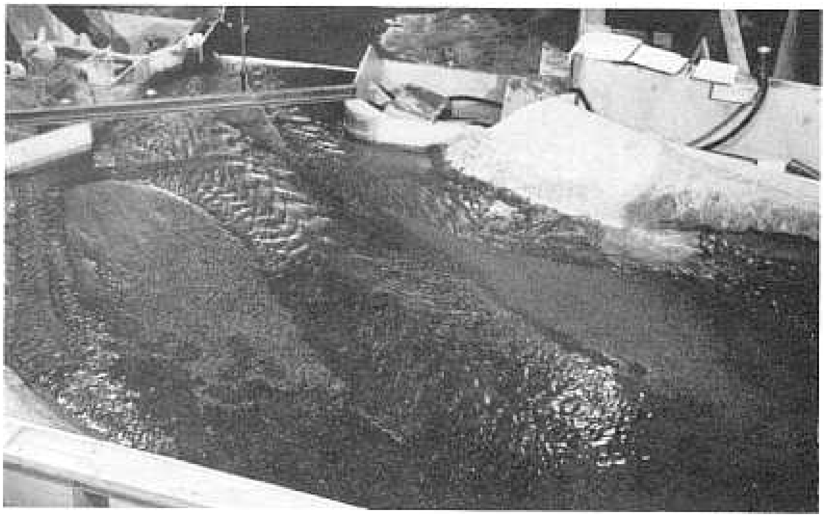


Figure 22.–Percent of tunnel sediment intake versus percent water delivery.

(a) Dry model bed before sluicing. P801-D-79628



(b) Flow of 8.5 m³/s (300 ft³/s) set before sluicing. P801-D-79629

(c) After 4 minutes of sluicing. P801-D-79630

Figure 23.–Time sequence photographs before, various times during, and after cut-bank sluicing.

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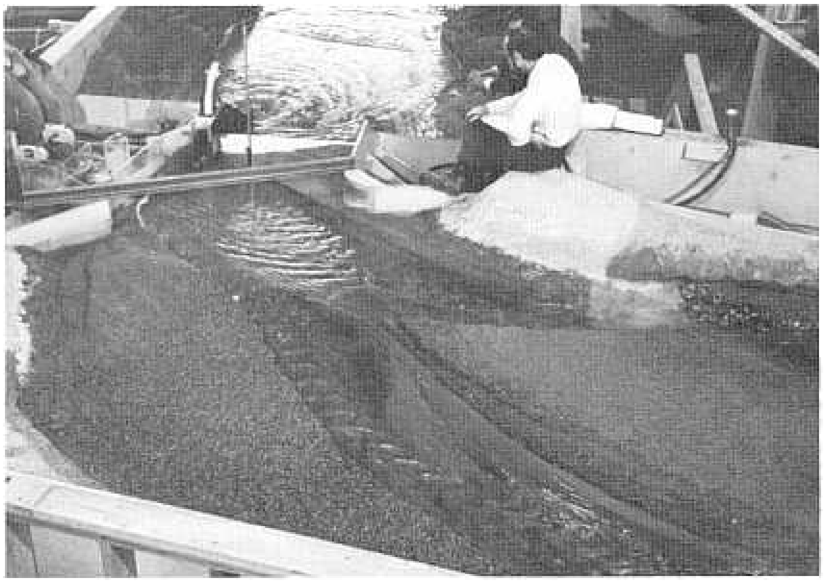
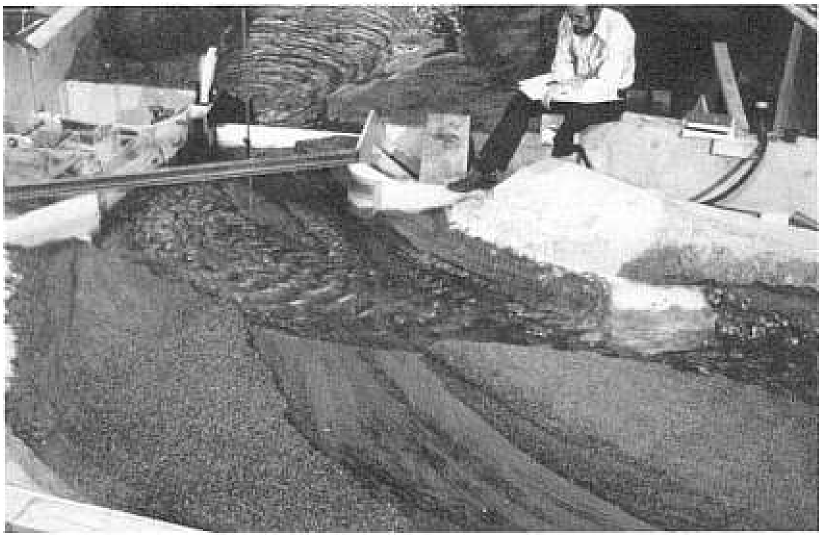


(d) After 4 hours and 44 minutes of sluicing. P801-D-79631



(e) After 17 hours and 24 minutes of sluicing. P801-D-79632

(f) Dry bed after 24 hours of sluicing. P801-D-79633

Figure 23.–Time sequence photographs before, various times during, and after cut-bank sluicing.–Continued

Slope Scaling

Bed slopes obtained during the equilibrium test averaged 0.01. Slopes determined from topographic maps for short reaches in the reservoir area varied from virtually flat to 0.0125, with an average of 0.0082. The Sedimentation Section used a slope of 0.012 and a Manning's "n" value of 0.05 for backwater and bedload calculations. These slope comparisons indicate that equation (5) is valid within the precision of measured slopes for the prototype and model.

Depth Scaling

The river depths determined by the Sedimentation Section are compared with the model measurements in figure 24. These plots show better agreement as riverflow increases. The model is 40 percent high at 100 ft^3/s (2.83 m^3/s) and 10 percent high at 700 ft^3/s (19.8 m^3/s). Some of the deviation was probably due to some reservoir ponding effect reaching the upstream end of the model.

Width Scaling

Model river widths were compared with estimated widths used by the Sedimentation Section and river widths measured at the project. It was concluded that width scaling could not be substantiated nor refuted because of the random nature of island and bar formations in the reservoir area. Reservoir river widths vary from station to station and, at given stations, vary with time and prior river width history, diversion, and sluicing operations.

Deposit Depths

During the hydrograph and equilibrium runs, sediment deposited on the model headworks floor to equivalent prototype depths of 1.2 to 1.7 m (4 to 5.5 ft). Field measurements indicated similar deposits of 1.1 to 1.4 m (3.5 to 4.5 ft).

The factors available for comparing conformance of time, bed slope, flow depth, and sediment depth in the structure indicated that the model scaled by Froude law for time and

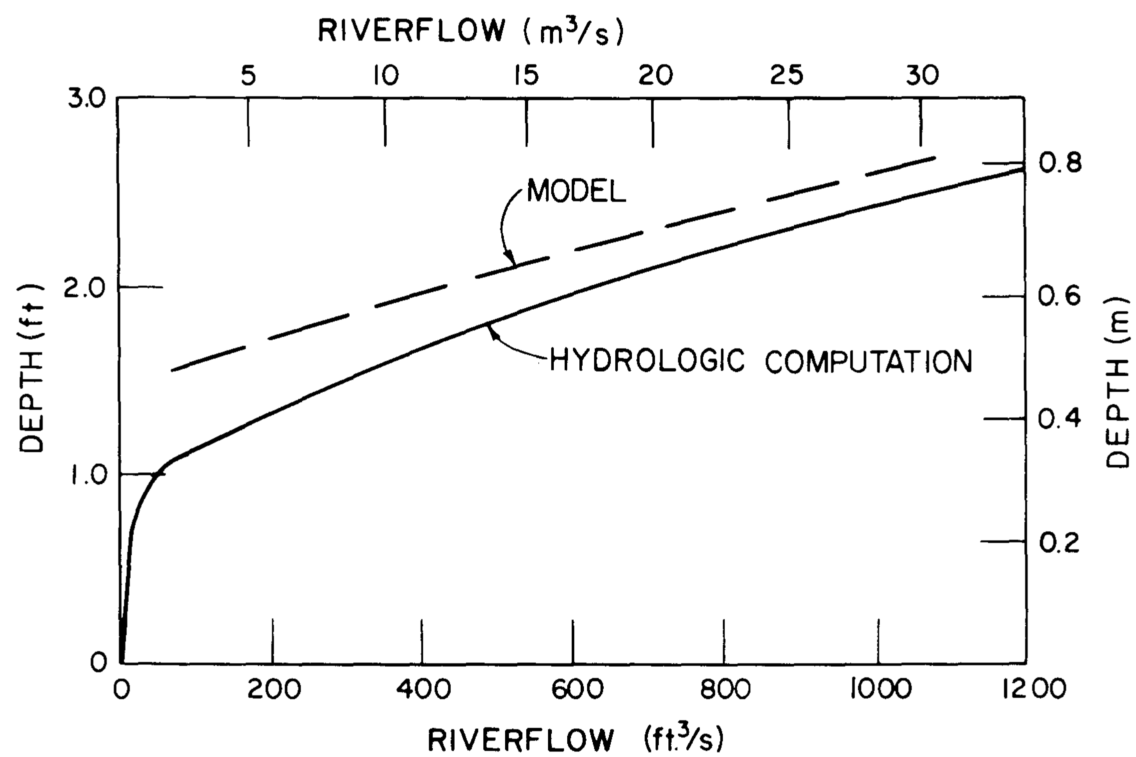


Figure 24.–Comparison of model and prototype river depth versus flow.

depth. Slopes of the model equaled the prototype without friction manipulation and operated without distorted dunes often encountered in models of fine sediment rivers. The model is considered by the author to be one of the least distorted sediment models ever developed by the Bureau.

MODIFICATION STUDIES

Relocated Modified Headworks

The first model modification tested, figure 25, included removing the 15.24-m (50-ft) ogee crest and replacing it with one pier and two radial gates. The headworks was extended to increase the trashrack area and reduce trash plugging and removal problems. To reduce the flow attack angle, the headworks inlet was also moved upstream and in line with the original river headworks convergence wall. A long, submerged training wall attached to the sluiceway formed a channel in front of the headworks. The channel, open to the river at the upstream end, was intended to provide more effective sluicing of sediment in front of the headworks. A long wingwall angled away from the upstream end of the headworks channel toward the left riverbank (fig. 26).

Model observation runs showed there was a strong tendency for deep flow along the wingwall to carry coarser bedload sediment. This coarse sediment traveled and accumulated in the upstream end of the sluiceway channel and piled up in front of the training wall (fig. 27). These deposits continued to grow and merged into one deposit of sediment passing over the wall and through the trashrack and traveling under the curtain wall into the original headworks (fig. 28). Continuous flow through the sluice gate did not prevent the buildup of the deposits. Intermittent sluicing with 8.5 m^3/s (300 ft^3/s) resulted in a deep sediment bed in the sluiceway channel because a sufficient transport velocity could not be generated by the remote sluice gate.

The remoteness of the modified headworks inlet from the spillway gates made sediment-moving difficult in the area in front of the training wall. Attempts were made to improve cleaning action in this area by adding another training wall extending upstream from the center of the spillway gate pier to increase the flow velocity. This wall did not help. Closing off the upstream end of the sluiceway channel did not prevent the sediment from going over the sluiceway wall and then into the headworks. It was noted that spilling small discharges over the top of the outer sluiceway wall into the channel caused strong spiral flow in the channel and increased the sediment transport rate over the capability of the sluiceway gate alone. The length of channel between the entrance and the sluice gate was still a problem because transport velocities could not be generated over the full distance.

Operation of the wide spillway gates disclosed another problem. Gate openings required to discharge water in excess of the tunnel diversion were often too small to pass the cobbles moving with the bedload material. Therefore, to reduce or eliminate the problem, a series of narrower spillway gates, the same width as the sluiceway gate, were installed in place of the wide gates (fig. 29).

Continuous Sluicing With the Sluiceway Gate and Two Narrow Spillway Gates

Sediment samples were obtained for riverflows of 14.7 m^3/s (520 ft^3/s) and 30-, 40-, 50-, and 60-percent water diversion to the tunnel headworks, with all excess flow passing through the existing sluiceway gate and the two spillway gates nearest the headworks. The percent of sediment delivery is plotted on figure 30 along with a replot of the curve from the existing diversion structure model (fig. 22) for comparison. The two curves indicate that the modification with the narrow gates on the spillway performed better than the existing diversion system for water delivery rates less than 45 percent. Above 45-percent water delivery, the narrow gates and existing headworks performed worse than the existing system, probably due to the increase in turbulence in front of the headworks. A Sedimentation Section study of the existing system indicated that for the years 1971 to 1975, the average amount of water diverted was about 62 percent. Using this value of water delivery and the dashed curve on figure 30, the narrow gate system was estimated to transport 93 percent of the sediment to the tunnel. For this scheme, an average of one-third more sediment would pass through the tunnel than in the existing diversion system.

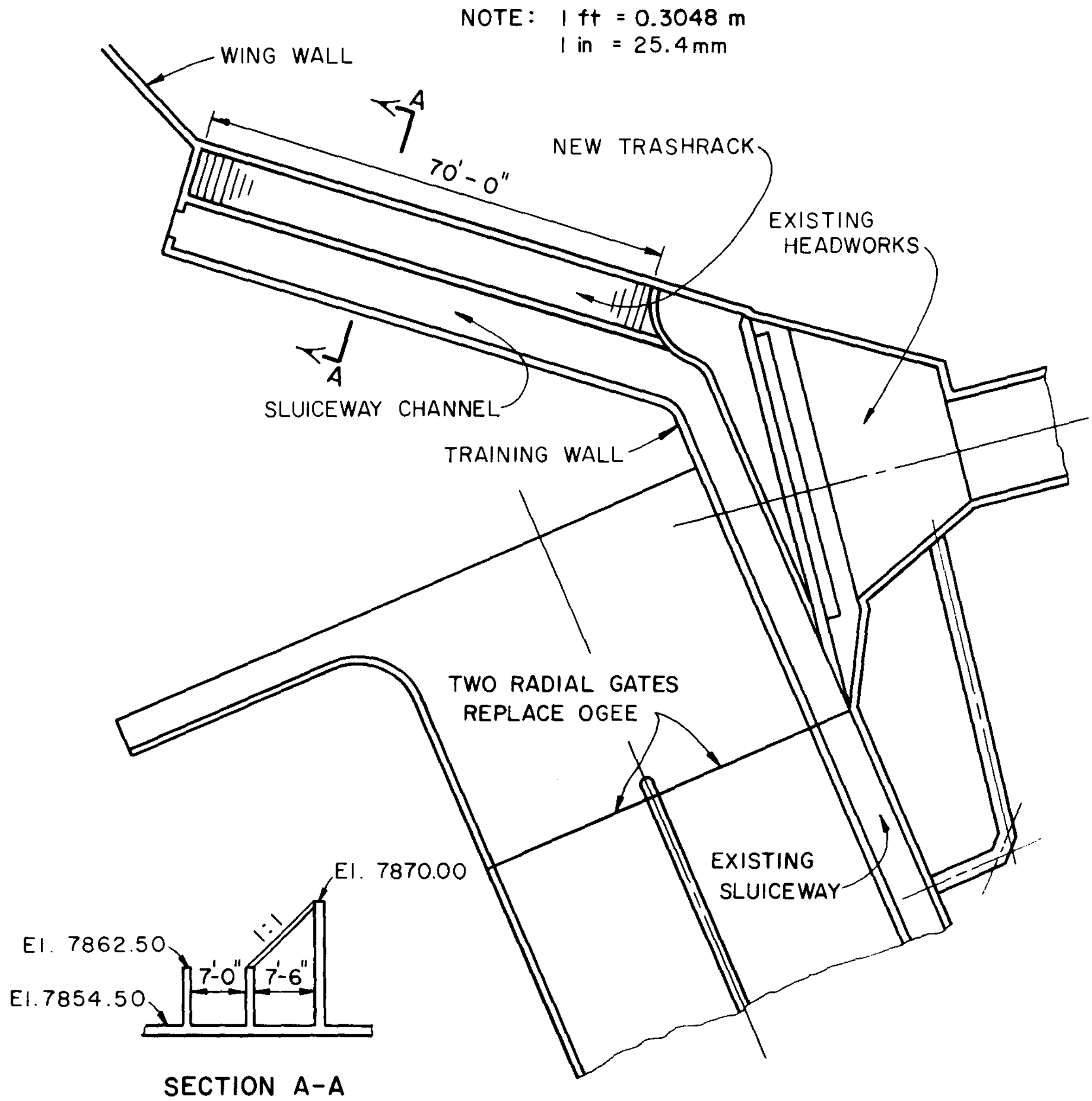


Figure 25.–Plan of first modification tested.

Figure 26.–View looking downstream at first test modification.
P801-D-79634

Figure 27.–Tunnel headworks and sluiceway of first modification.
P801-D-79635

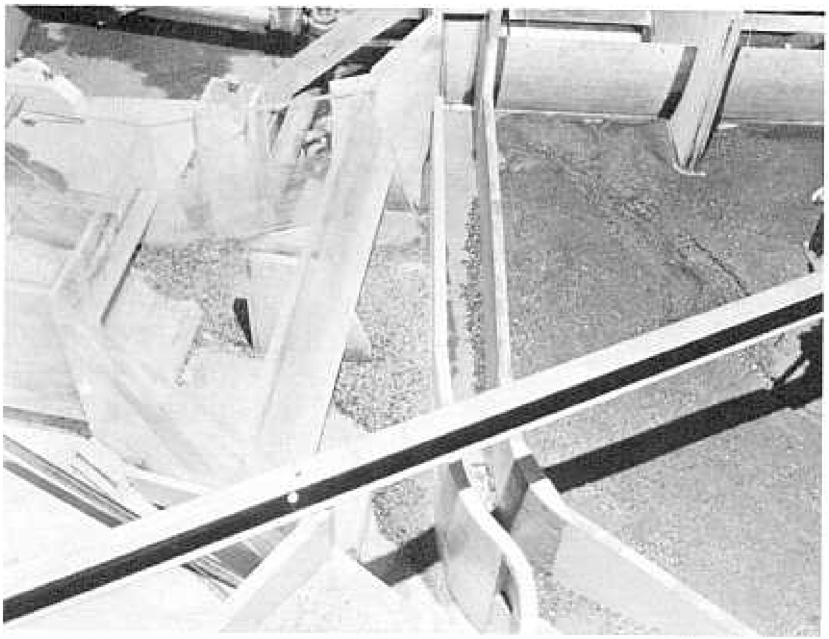


Figure 28.–Sediment that traveled into the original headworks during operation with the first modification. P801-D-79636

Figure 29.–Narrow spillway gates in conjunction with the existing headworks. P801-D-79637

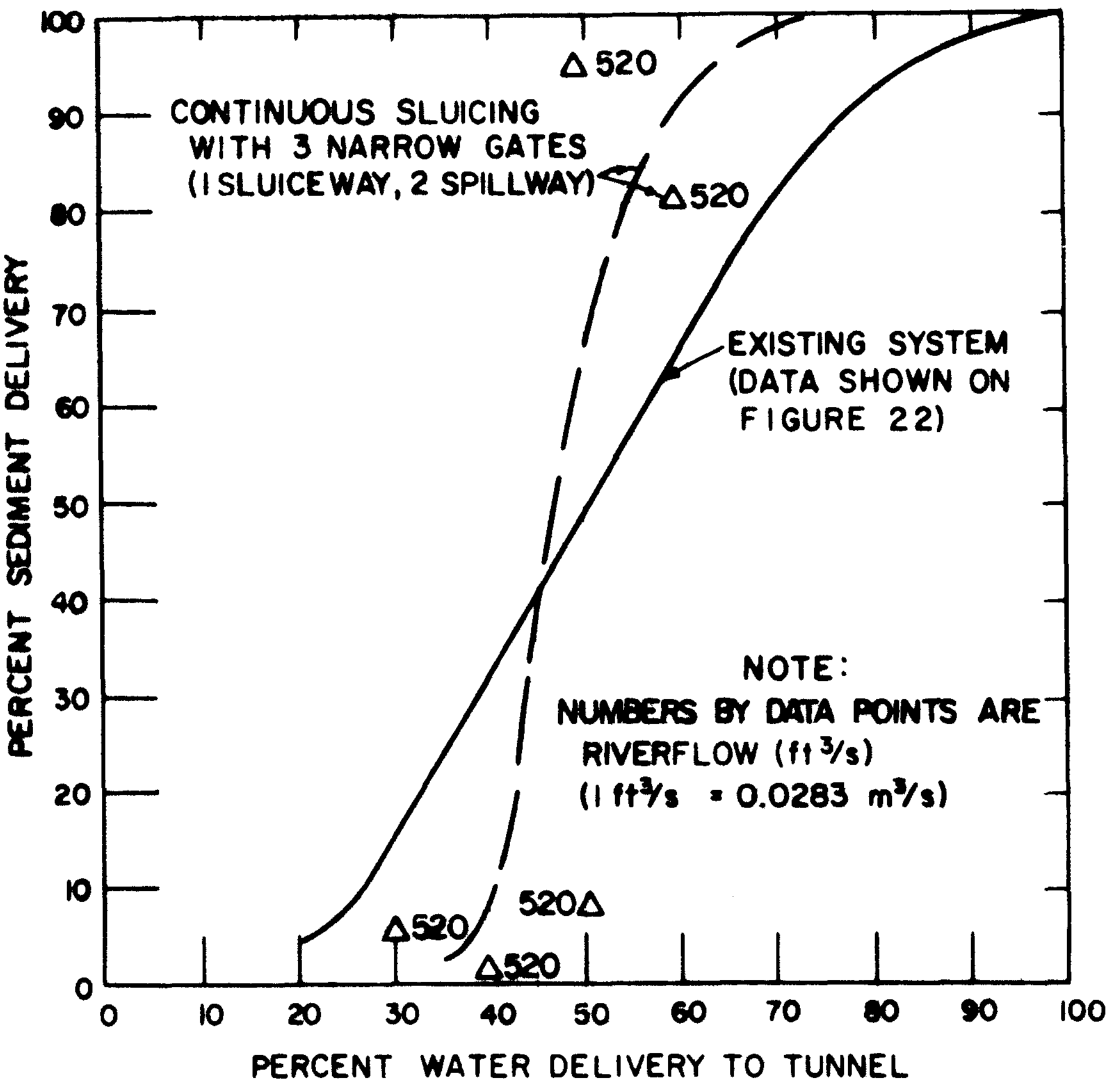


Figure 30.–Percent sediment delivered to tunnel versus percent water delivery for the existing and narrow spillway gate sluicing systems.

Development of Recommended Modification Design

Tests of sluicing over the top of short-length trap walls directly in front of the original tunnel headworks indicated possible advantages. Up to four parallel traps were studied in this series of tests. A schematic view of this flow process for one trap is shown on figure 31. Water falling into the 1.5-m (5-ft) wide trap caused a strong single spiral flow toward the sluice gate and was very effective in removing sediment. As the flow plunged over the wall, it formed a trough between the trashrack and the outside wall of the downstream trap, the trash floated back away from the trashrack, and was carried by the water under the sluice gate.

Sluicing over the top of the trap walls did not transport sediment into the headworks by deep scouring action in the reservoir area. Rather, the sediment was skimmed from the surface of the reservoir bed.

An advantage of the traps would be that sluicing could be done without closing the tunnel headworks gate, as is required with the existing structure. Also, because of the trap wall and gates in the spillway outside of the traps, the trashrack could be dewatered for inspection or

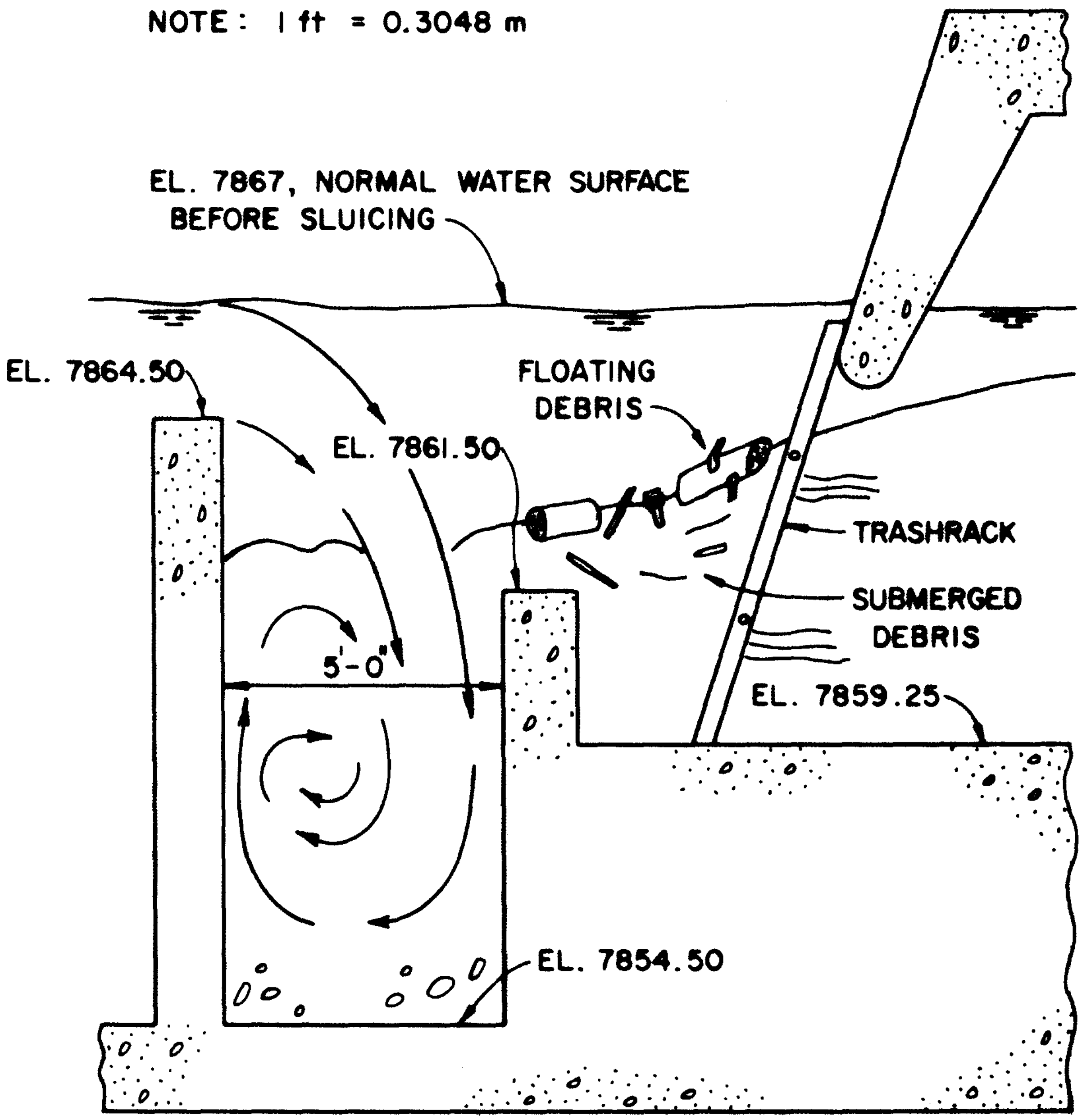


Figure 31.–Schematic sketch of overwall trap sluicing.

maintenance during flood stage by lowering the reservoir level. The sediment dropslot pipe could also be unwatered and cleaned if necessary.

If necessary, water relatively free of sediment could be passed through the traps to reduce the sediment average concentration cleaned from the traps or to help move sediment downstream. If the traps are kept clean and functioning, then there would be a reduction of sediment size and quantity going into the tunnel, resulting in a reduction of tunnel invert abrasion.

In an effort to reduce the number of walls and sluice gates and the cost of construction, one 1.52-m (5-ft) and two 3.35-m (11-ft) wide traps were tried for overwall sluicing capability. Operation with the wide traps showed that they could not be effectively cleaned because there was no longer a strong single spiral flow and downstream velocity toward the gate.

Scour holes at the upstream end of the first modification sluiceway channel and existing headworks, figures 27 and 29, suggested the possibility that successive spurs might force sediment toward the reservoir outflow through the spillway gates, rather than leaving a deposit in front of the headworks. Therefore, several combinations of traps and spurs on the approach wall and the front trap wall were tested, such as shown in figure 32. The spurs acted like bottom vanes for short periods at times when the flow was parallel to the headworks approach wall. However, they generally became buried and ineffective (fig. 32b). Sediment measurements indicated no significant difference in the tunnel sediment intake for traps with or without spurs. During this stage of the studies, it was concluded that two high-wall bays plus the sluiceway bay, having widths of 1.52 m (5 ft), were the minimum number of traps that could be used. Therefore, the remaining tests were directed toward the recommended system shown in figures 33 and 34.

Types of Tests With Proposed Modification

Two types of sampling tests were made with the recommended modification. One was tests terminating with the traps in various degrees of fullness but still functioning. Wet sieve sediment samples and quantities of the sediment depositing in the traps and on the headworks floor were used to compute the sediment ratios.

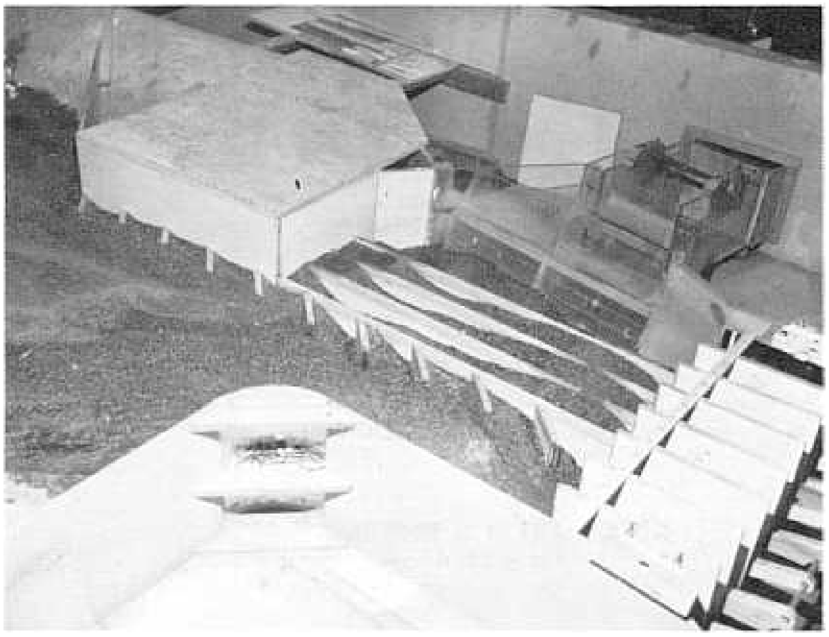
The second type consisted of hand-forming a delta-front bar higher than naturally formed by flow over the traps to the tunnel headworks. The model was then operated at constant discharge, scouring the formed bed down to transport equilibrium bed level related to the river discharge and flow division being tested. This operation represented the worst possible situation – when the traps would be left unattended and not sluiced.

Sediment Division

Sediment samples were taken to compute the percent of sediment delivered to the tunnel. Sediment percentages were plotted versus water delivery percentages for both types of tests. There was considerable scatter of data caused by variable river approach conditions, different trap volumes accumulated, and the time when sediment samples were obtained. Upper bounds from 15 data points for the traps functioning and from 10 data points for traps full are plotted as dashed lines on figure 35 along with replots from figure 30 of the curves of the existing diversion system and continuous sluicing with three narrow gates. The upper bounds represent worst performance. Bands of data for both cases overlapped.

Using the 62-percent average water delivery determined by the Sedimentation Section and curves on figure 35, results in an estimated average sediment delivery to the tunnel of: 93 percent for continuous sluicing with narrow spillway gates and existing headworks system (fig. 29); 68 percent for the existing system (fig. 23); 30 percent for narrow gates, existing headworks, and full traps; and 13 percent for narrow gates, existing headworks, and traps functioning (fig. 33).

Figure 35 indicates that with the traps functioning the proposed modification excludes sediment from the tunnel better than the existing system for all water delivery percentages. Full traps exclude sediment better than continuous sluicing with the no-trap, narrow-gate system for deliveries greater than 42 percent, and better than the existing system for up to 73-percent water delivery. Continuous sluicing with the



(a) Four 1.52-m (5-ft) wide sluicing traps and spur walls in reservoir. P801-D-79638

(b) Three 1.52-m (5-ft) traps with spur walls. P801-D-79639

Figure 32.–Some sluicing-trap and spur-wall arrangements tested.

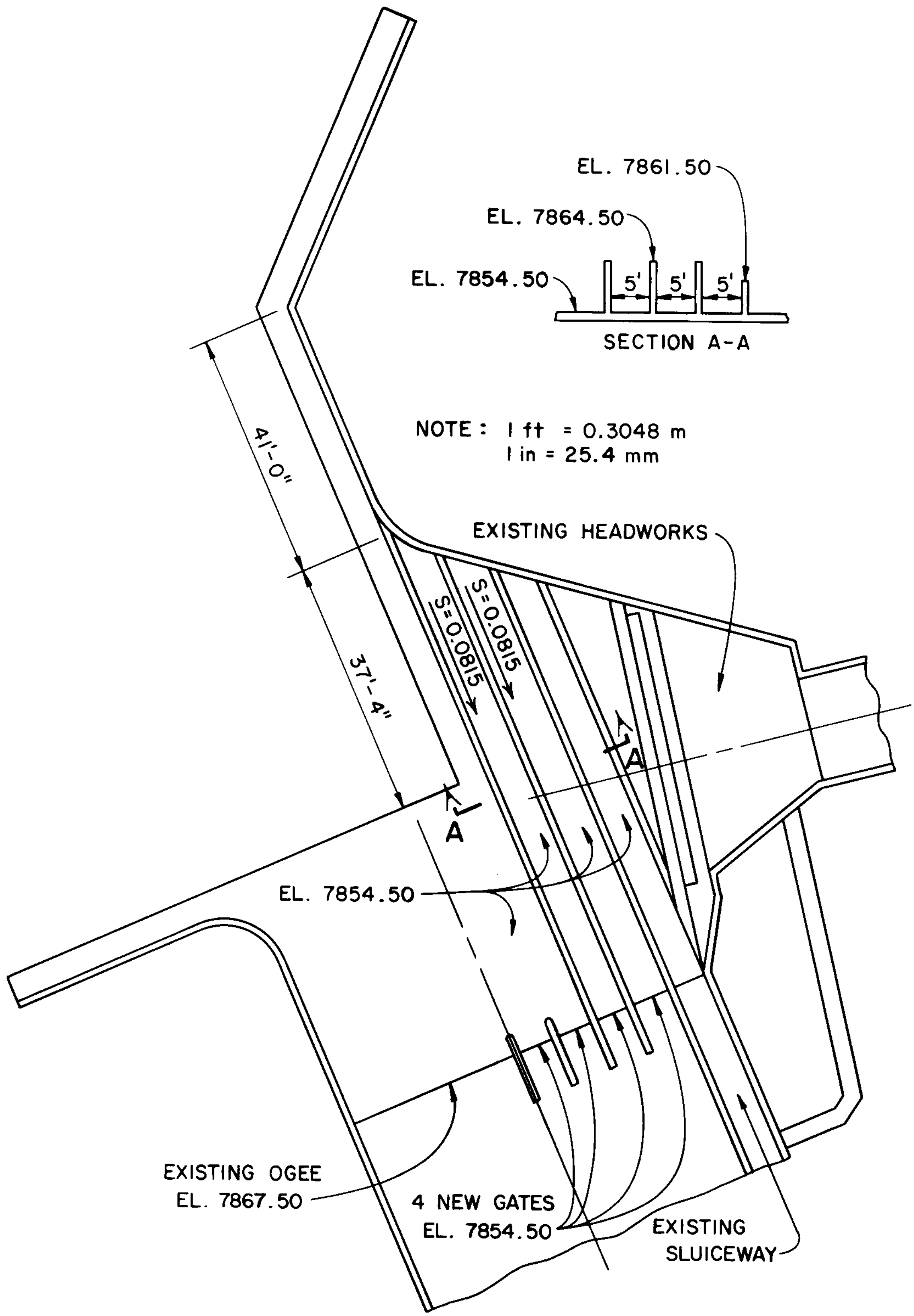


Figure 33.–Plan of recommended modification.

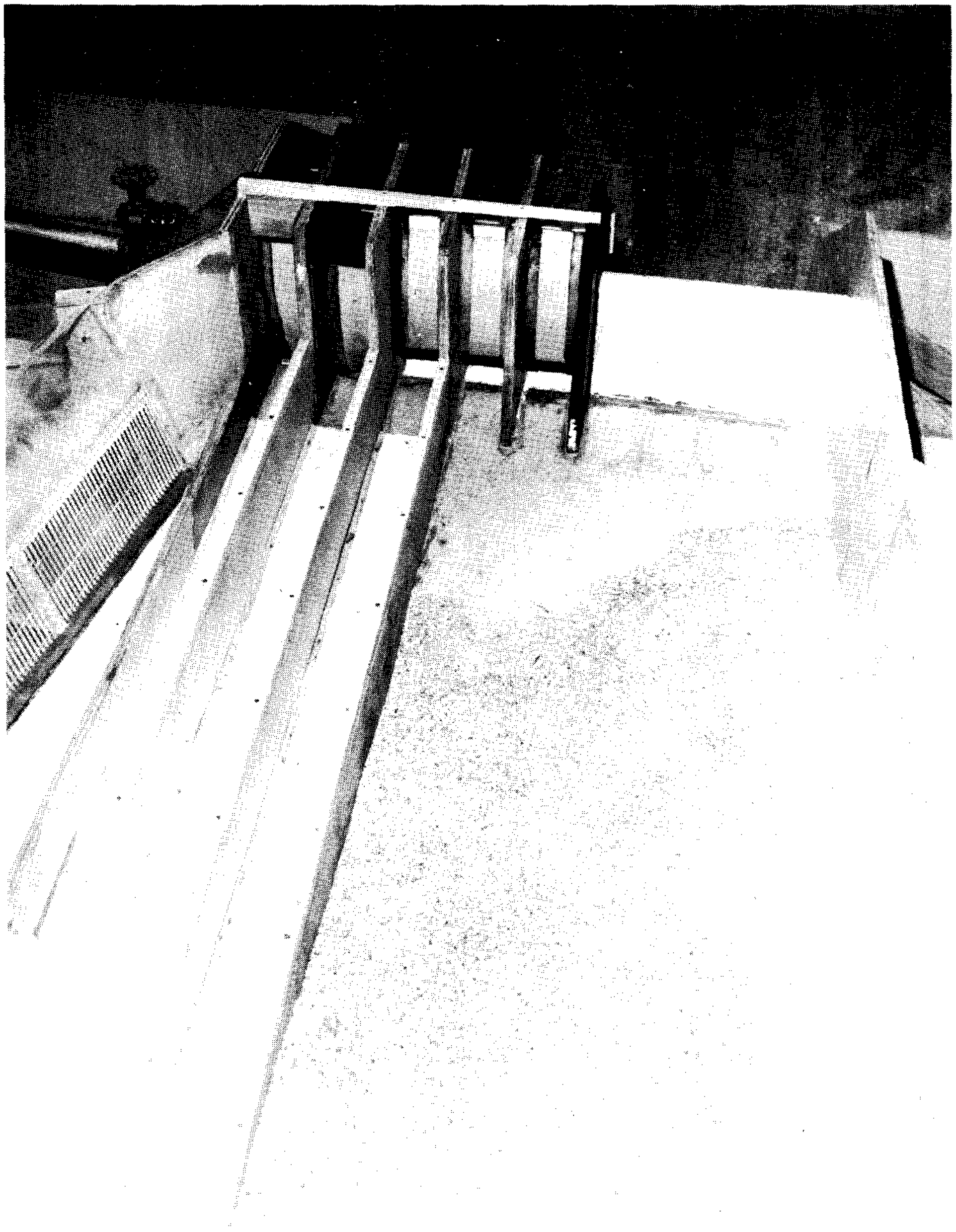


Figure 34.–Sediment traps of the recommended modification. P801-D-79640

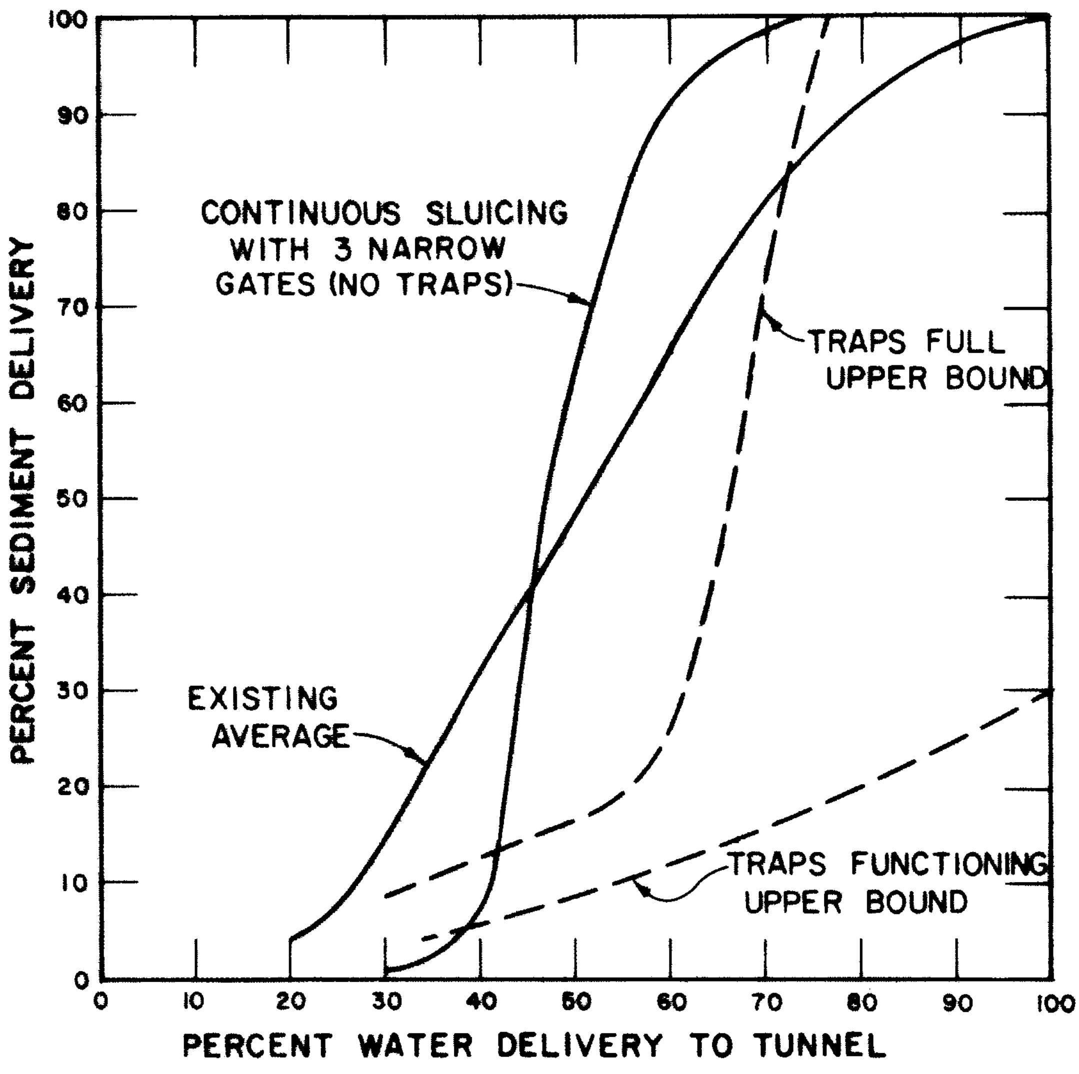


Figure 35.–Sediment delivered to the tunnel for various water deliveries and sluicing schemes.

no-trap, narrow gate system was better than the existing system for delivery percentages less than 45 percent.

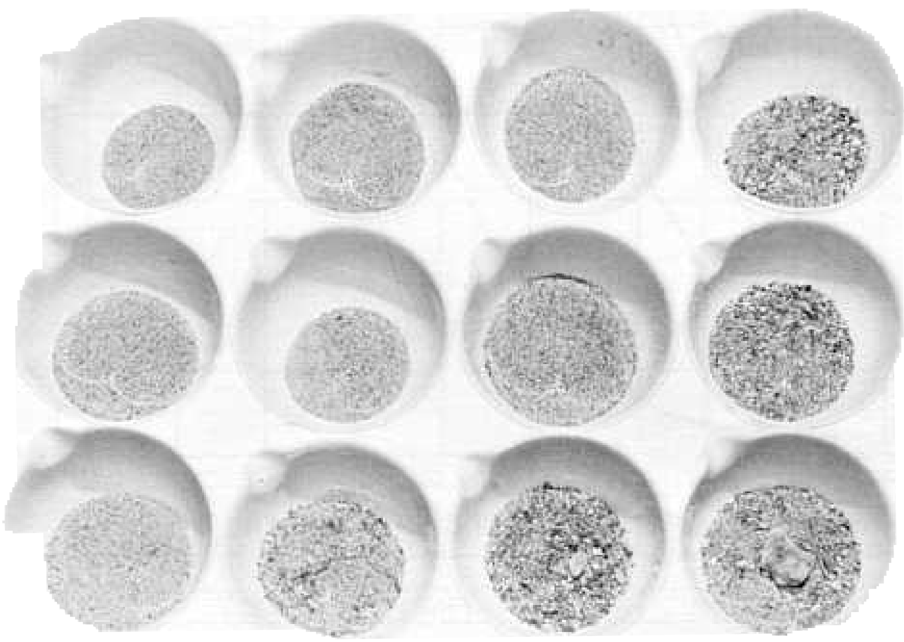
Sediment Size Segregation

As expected, the sediment became increasingly finer in successive downstream traps toward the tunnel headworks. Figure 36 shows samples taken from the traps and the headworks floor with respect to the sample location and flow rate. This photograph clearly shows the segregation of sediment in the traps and headworks.

The grain analyses of the headworks floor material shown in the photograph for three flows are plotted on figure 37, for comparison with a sample from the riverbed material. These gradation curves show the size segregation from the average bed material outside of the traps to the headworks floor deposits. The effectiveness of the traps on the size segregation diminishes as turbulence at higher flows increases. At 14.2 m³/s (500 ft³/s), the ratio of the 50-percent sizes for the bed materials to that on the headworks floor was 20:1 and at 31.7 m³/s (1120 ft³/s), the ratio was 5:1.

Suggested Maximum Water Delivery

Suggested percentage of water delivery versus riverflow is shown on figure 38. The 60-percent water delivery to the tunnel for riverflows from about 8.5 to 25.5 m³/s (300 to 900 ft³/s) was selected because the amount of sediment delivered to the tunnel increases rapidly from 20 to 95 percent between 56- and 75-percent water delivery (fig. 35). Larger percent water delivery for the smaller riverflows is considered permissible because the transport of river sediment decreases rapidly with decreasing riverflow. At higher flows, the excess of the design flow of 14.7 m³/s (520 ft³/s) is all



14.2 m³/s (500 ft³/s)

19.8 m³/s (700 ft³/s)

32.0 m³/s (1120 ft³/s)

0 100 200

SCALE IN mm

Headworks Sill | Downstream Trap | Middle Trap | Upstream Trap

Figure 36.–Trap and headworks floor sediment samples showing segregation of grain size with distance toward tunnel headworks and with flow. P801-D-79641

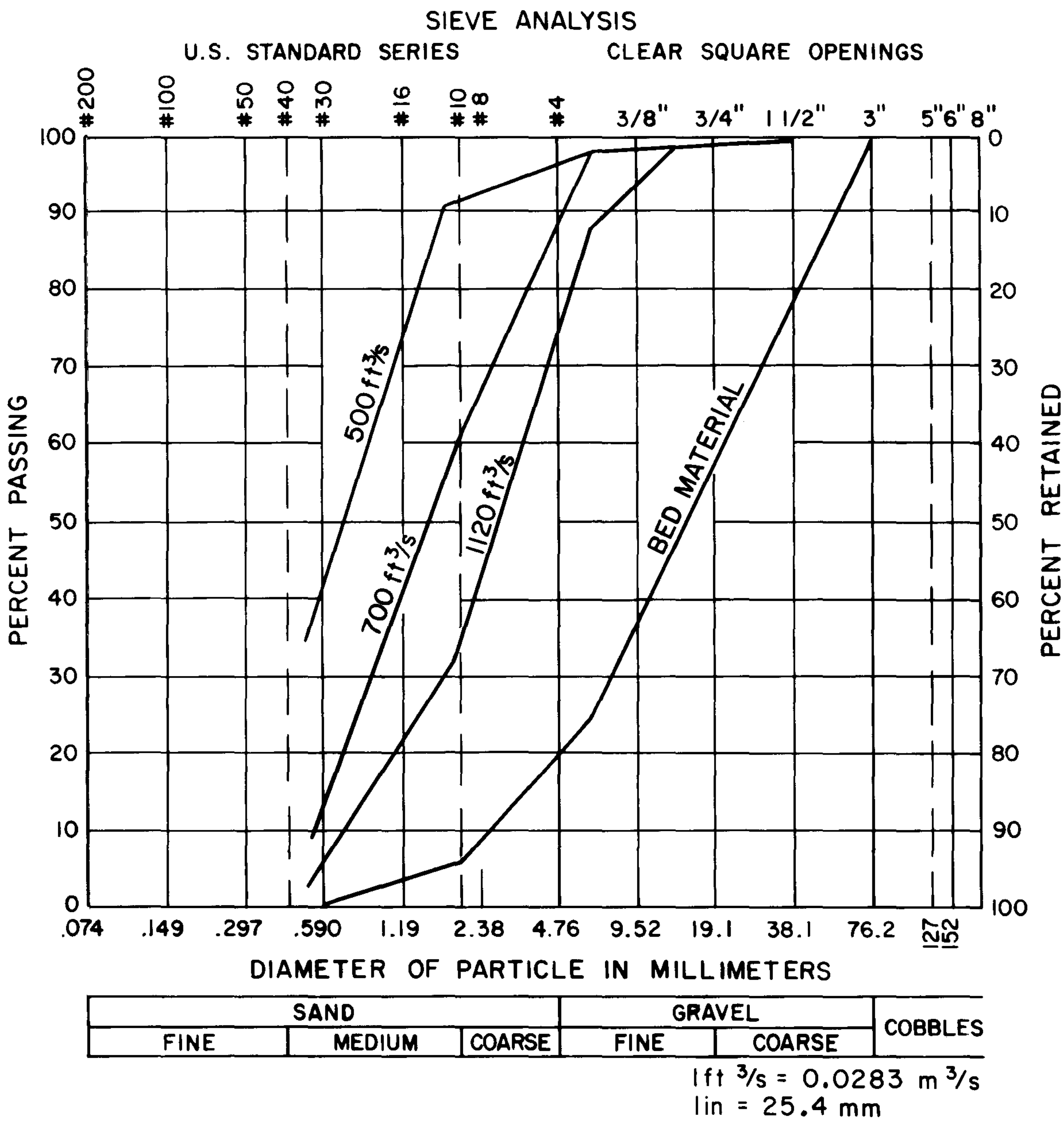


Figure 37.–Comparison of headworks floor material with bed material deposited at various riverflow rates.

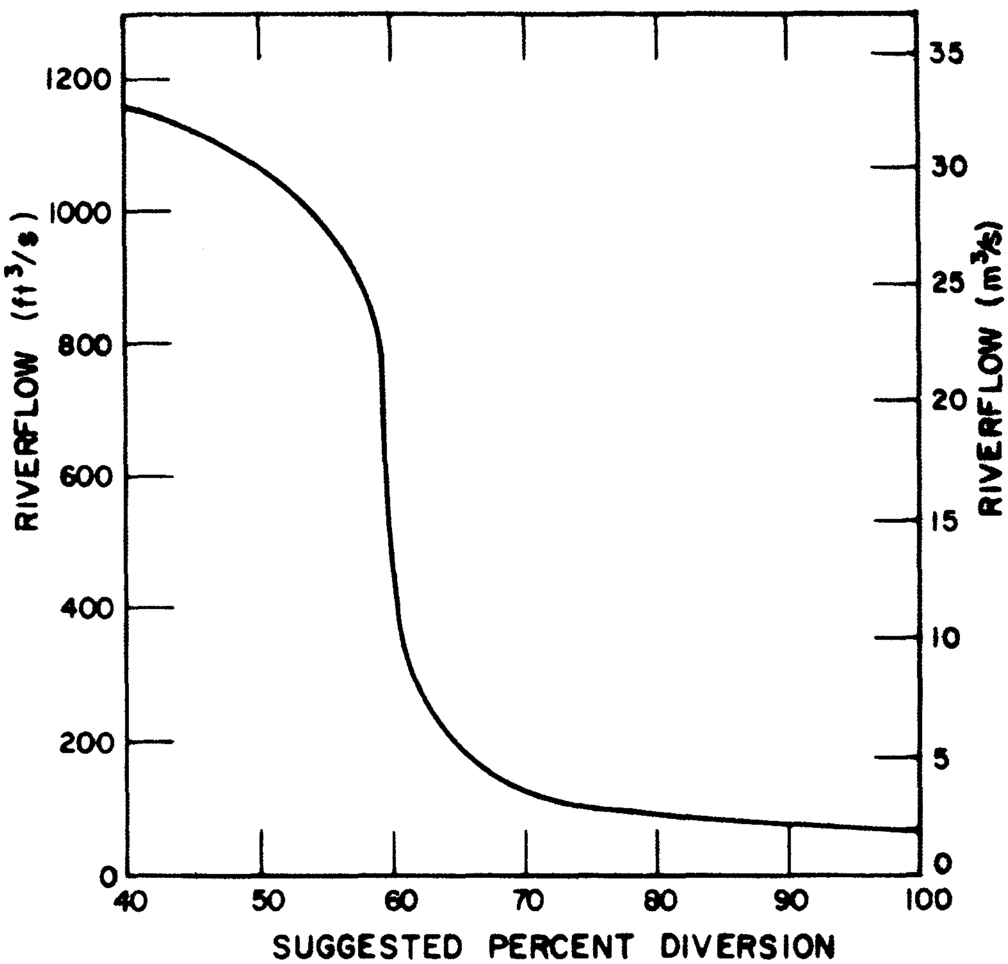


Figure 38.–Suggested percentage of riverflow to be diverted.

available for in-front-of-trap bypass flow and the percentage water delivery is generally well below the point where sediment increases rapidly (fig. 35). The operators may want to, or have to, deviate from the suggested operating curve, but they should remain cognizant of the reasons for its shape.

TRAP SLUICING

General

Sluicing of traps was performed at riverflows of 8.5, 14.2, and 32.0 m³/s (300, 500, and 1130 ft³/s). These tests indicated that sluicing of the traps becomes increasingly difficult as the riverflow increases, even with the capacity of the gates outside of the trap bays. For large flows, excess surface water flowed over the top of the traps toward the tunnel headworks, back out, and then under the outside spillway gates. This return flow weakened the vortex in the outer trap during overwall sluicing. Also, large riverflows carry larger quantities of sediment, slowing the cleaning action. For example, from figure 11, a 32.0-m³/s (1130-ft³/s) discharge carries about six times as much sediment as the 14.2-m³/s (500-ft³/s) flow. Trap cleaning with the larger flow takes about 2-1/4 times longer than with the smaller discharge. For deep deposits on top of the trap wall and also for larger flows, cleaning was better when the gates of the two outer traps were opened at the same time to start the cleaning action.

The longest time required to clean a single trap was about 15 prototype minutes. The shortest time was 1 minute for a downstream trap with a small peak of sediment near the middle of the trap. It is estimated that it would take from 36 to 60 minutes, including gate opening and closing times, to clear the prototype traps.

Estimating When to Clean Traps

In the field, the decision of when to clean the traps will be difficult to make by visual observation. By experience, however, operators may become aware of water surface wave action that indicates the need for cleaning the traps. The model indicated that the best time to clean traps was soon after the sediment reaches the top of the middle trap. For minimal tunnel sediment intake, sluicing will at times be required both night and day. If only remote sensing of flow and flow control are used, then the decision of when to sluice will have to be estimated using figures 11 and 35. Experience gained during operation may substantiate the model results and subsequently indicate proper cleaning time. However, it might be possible to set up a sounding or ultrasonic depth-measuring system to determine when sediment accumulation warrants a sluicing of the traps.

Effective Two-Trap Capacity

The volume of sediment held in the traps, as estimated from photographs, varied with riverflow rates. This was because of variation of flow approach direction; deposit shape and location within the traps; and the sediment lift height and jump distance due to turbulence, which varied with the flow rate. Thus, the general shape of the curve (fig. 39) can be considered the result of the interaction of flow turbulence,

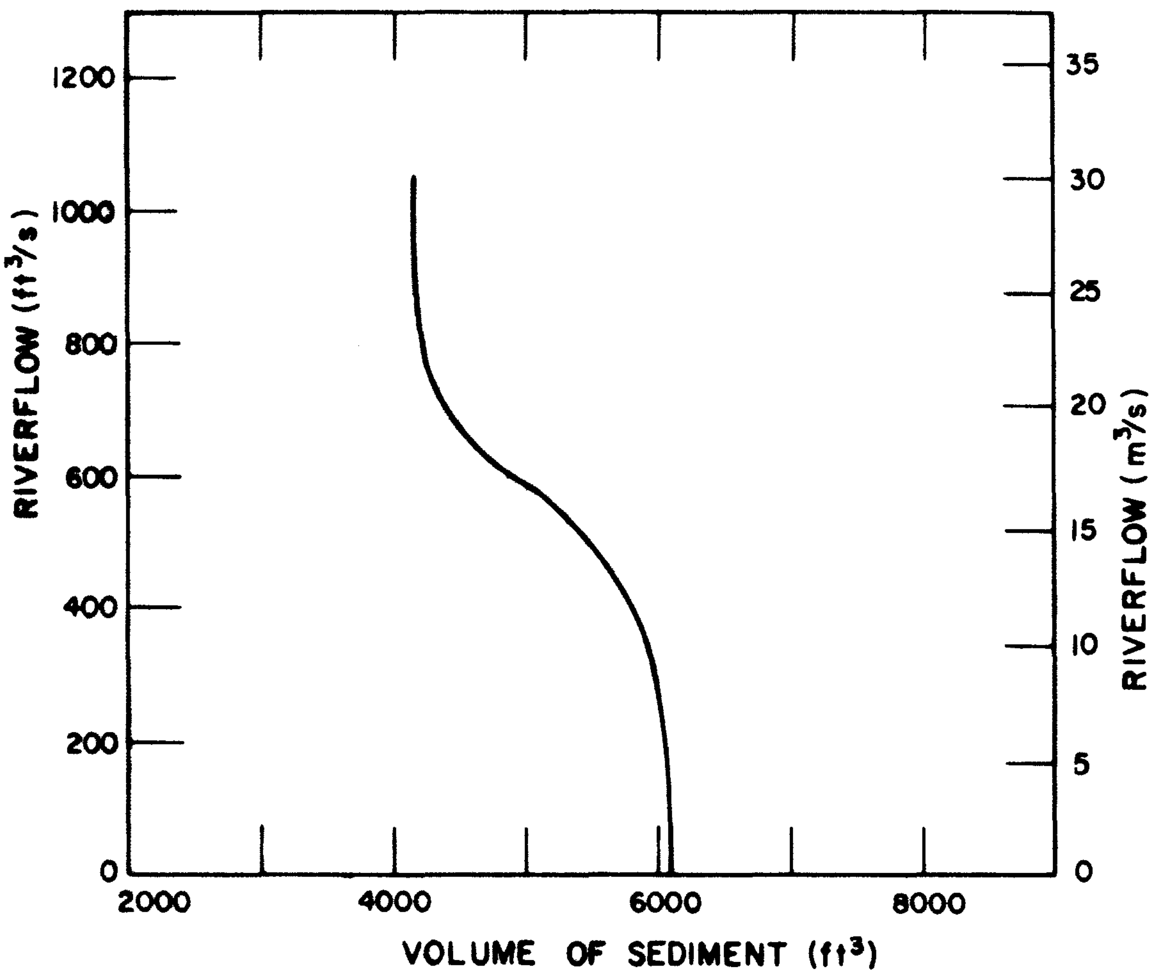


Figure 39.–Effective two-trap volume versus riverflow.

trap geometry, deposit shape, and water available for in-front-of-trap bypass flow.

Trapping Efficiency

The percentage of river sediment being trapped varies with riverflow rate, tunnel delivery rate, and the amount of in-front-of-trap bypass flow. The curve on figure 40 was determined from the model for the range of expected flow rates.

Time to Fill Two Traps at Constant Flow Rates

The sediment loads from figure 11 and the percentage trapping rate from figure 40 were used to determine the filling rates for the two traps. To convert sediment load from mass to volume occupied, it was assumed that the dry mass density of sediment deposits was 1600 kg/m³ (100 lb/ft³) and had 40-percent porosity. Time to fill two traps with constant riverflow was computed and plotted on figure 41. This curve shows how rapidly the trapping rate increases for riverflows above 14.2 m³/s (500 ft³/s). The curve on figure 41, however, is useful only for periods of relatively constant flow, as long as or longer than those shown on the plot. Therefore, it will usually be better to estimate overwall sluicing intervals for a more dynamic situation, such as during sequences of flood hydrographs.

Estimating Sluicing Frequency

For computation of sluicing frequency, hydrographs should be put into a flow-rate step form at some convenient time interval, and a curve for time rate of sediment trapped versus river discharge determined from figures 11 and

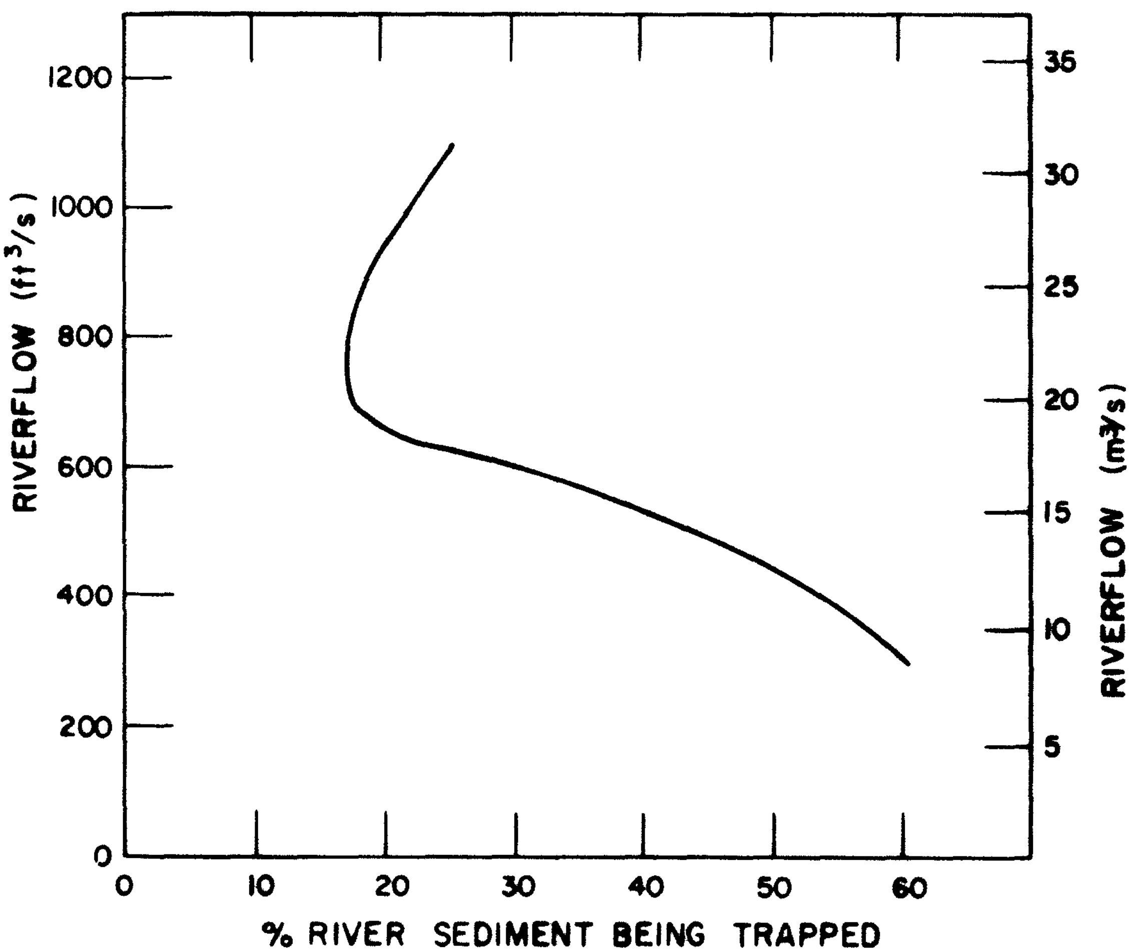


Figure 40.–Percent sediment trapped versus riverflow.

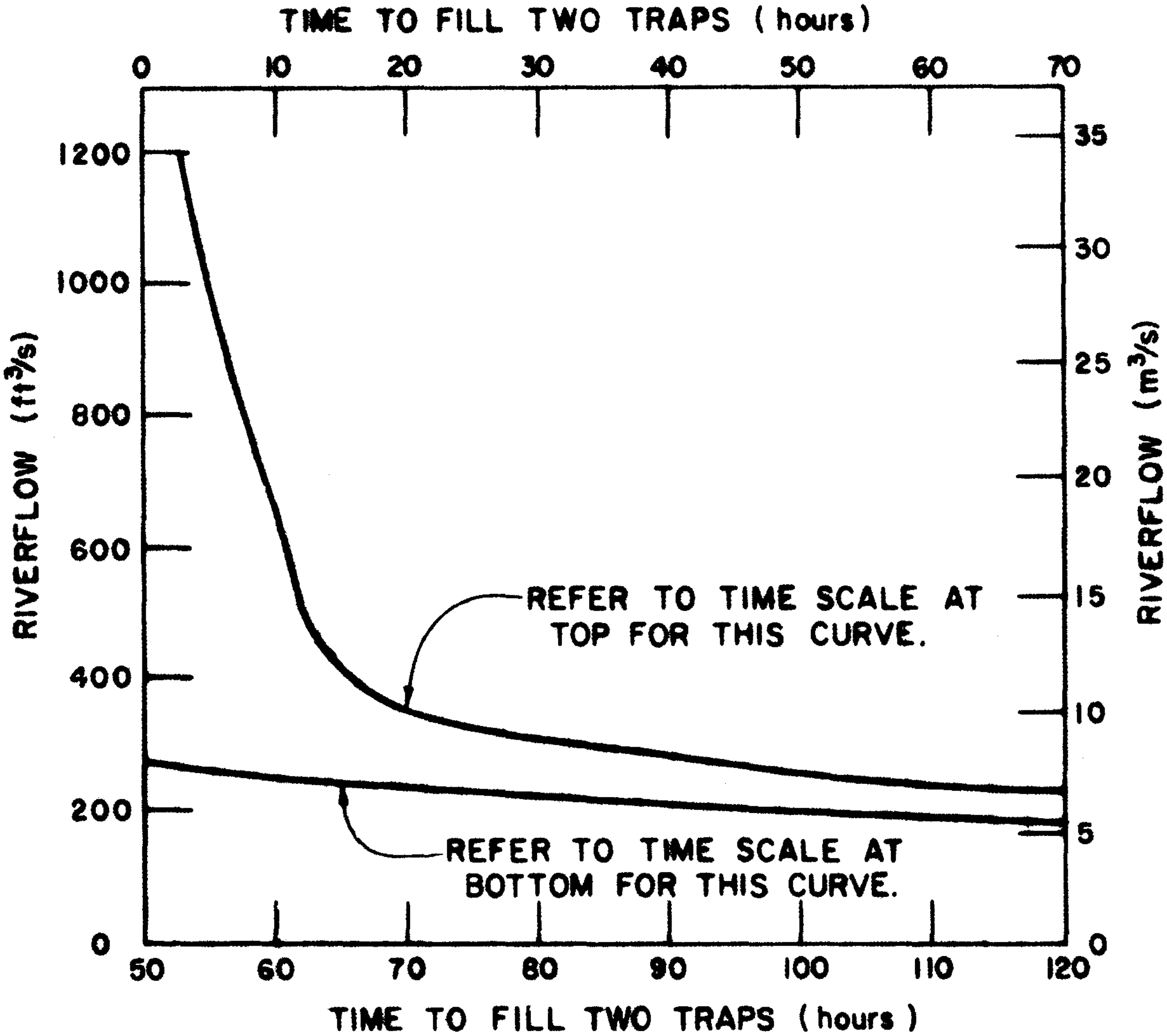
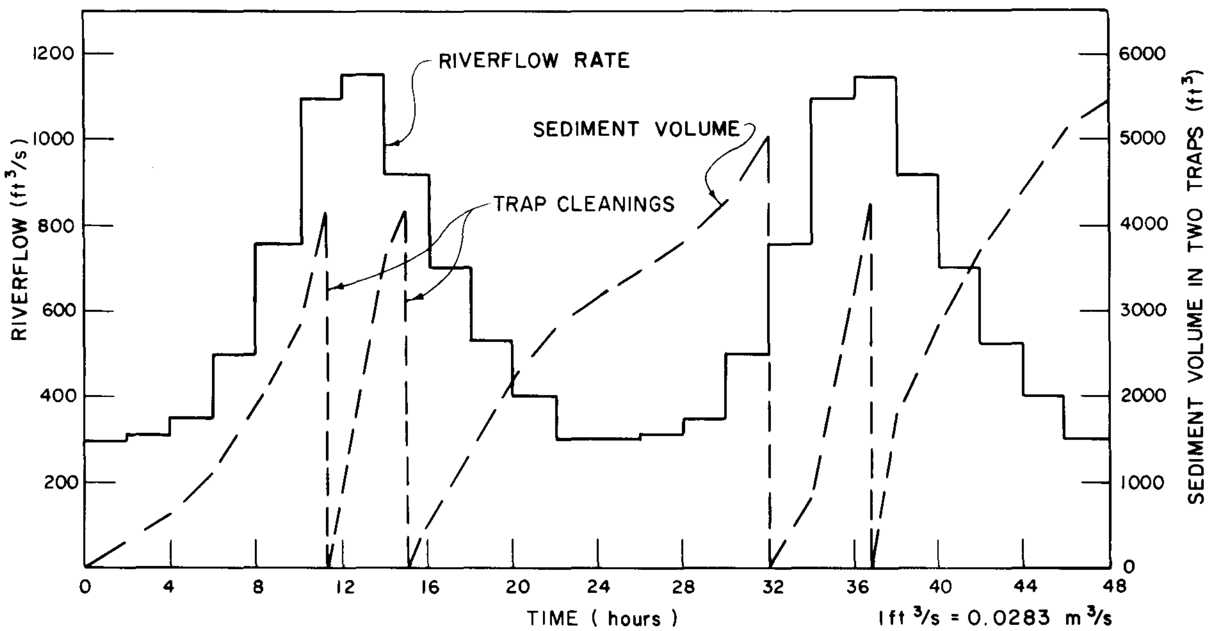
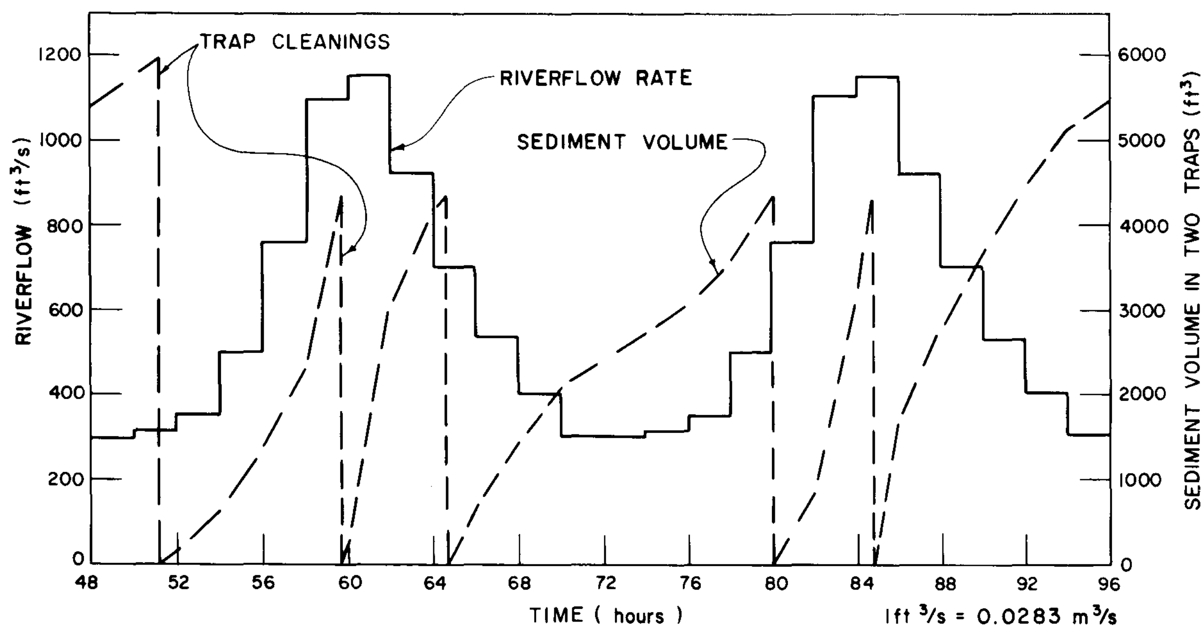


Figure 41.–Time interval between trap sluicing for constant riverflow.

40. From these curves, accumulated volumes at the end of each flow step are determined. When the accumulated volume and flow fall to the right and above the curve on figure 39, then the traps should be cleaned. This type of analysis was done with a series of step hydrographs (fig. 42a and 42b) that simulate the hydrograph for June 9, 1973, the worst of five submitted by the project office. The volume of sediment accumulated is also plotted on figures 42a and 42b as dashed lines. The vertical drops represent trap cleaning as dictated by the volume limits of figure 39. This analysis indicates that for minimal tunnel sediment intake, trap cleaning should be done twice during the higher portions of the hydrograph. The procedure outlined for estimating when to clean the traps can be refined by accounting for the amount of time required to complete each sluicing operation. Using partial, rather than total, cleaning of the traps might be expedient during the more difficult higher discharges.



(a)



(b)

Figure 42.–Step hydrographs and computed trap volume showing sluicing times.

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