UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF THE SPILLWAYS AND OUTLET WORKS--GLEN CANYON DAM COLORADO RIVER STORAGE PROJECT, ARIZONA
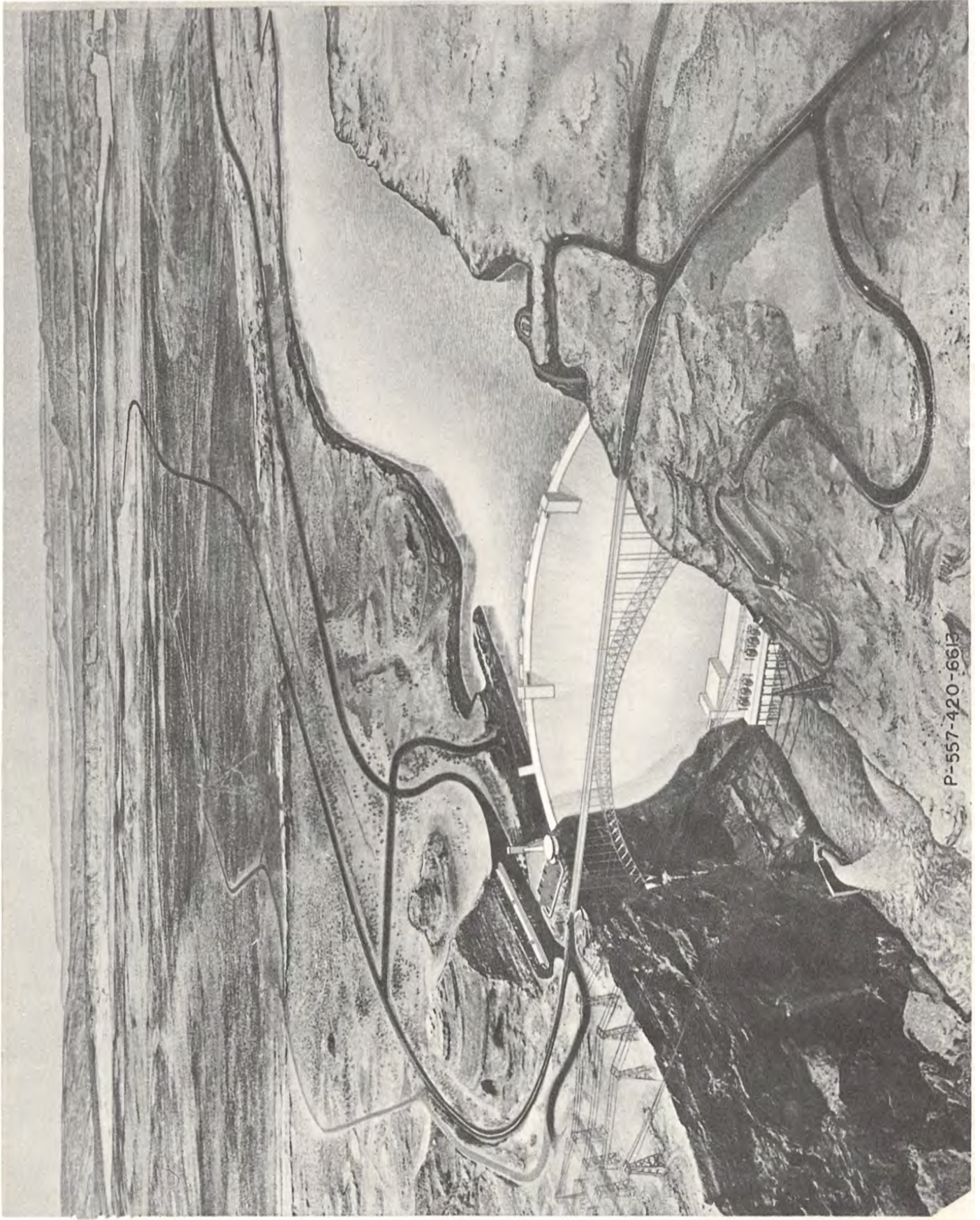
Report No. Hyd-469

Hydraulics Branch
DIVISION OF RESEARCH
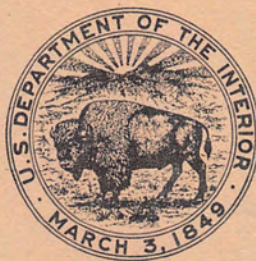
OFFICE OF CHIEF ENGINEER
DENVER, COLORADO

February 18, 1964

P-557-420-6613

CONTENTS

CONTENTS--Continued

CONTENTS--Continued

CONTENTS--Continued

ABSTRACT

Hydraulic model studies were performed to investigate flow conditions in the diversion works, the tunnel spillways, and the river outlet works. The alinement of the tunnels was satisfactory for both diversion and spillway flows. A low, curved concrete wall placed adjacent to the right canyon wall will protect the canyon wall from undermining and erosion damage by diversion flows. The spillway approach channels were greatly reduced from their original size. Flow through the crest sections was excellent and no adverse pressure conditions were noticed. However, the preliminary tunnel transition was too abrupt as indicated by rough flow conditions and subatmospheric pressures. A longer, adequately streamlined transition was developed for prototype construction. Flow in the 41-foot-diameter tunnels was excellent at all discharges. The preliminary rectangular flip bucket at each downstream tunnel portal was replaced by a bucket in which the circular invert of the tunnel intersected the vertical curve of the bucket. This type of bucket eliminated the need for a circular-to-rectangular transition. The outside walls of both buckets were turned inward to direct the flow into the river in a more favorable pattern. Pressures as great as 211 feet of water were measured in the invert and on the wall of the left bucket. The river outlets were arranged in a fan shape to reduce their erosive tendencies in the river channel. Tailwater drawdown tests indicated that the tailwater elevation at the powerhouse will be as much as 30 feet lower than the downstream water level.

DESCRIPTORS--*hollow jet valves/ *radial gates/ afterbays/ diversion tunnels/ *flip buckets/ hydraulic structures/ uplift pressures/ cavitation/ control structures/ discharge coefficients/ flow/ Froude number/ head losses/ *hydraulic models/ jets/ Manning formula/ open channel flow/ spillway crests/ surges/ tailrace/ piezometers/ pressure measuring equipment/ energy losses/ negative pressures/ stream erosion/ backwater/ drawdown/ transitions/ training walls/ velocity/ velocity distribution/

IDENTIFIERS--approach channel/ tunnel transitions/ tunnel spillway

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Office of Chief Engineer Report No. Hyd-469
Division of Research Author: T. J. Rhone
Hydraulics Branch Reviewed by: W. E. Wagner
Denver, Colorado Submitted by: H. M. Martin
February 18, 1964

HYDRAULIC MODEL STUDIES OF THE SPILLWAYS
AND OUTLET WORKS--GLEN CANYON DAM
COLORADO RIVER STORAGE PROJECT, ARIZONA

PURPOSE

The studies were conducted to thoroughly investigate the hydraulic
characteristics of the tunnel spillways and river outlet works to
provide reliable performance under all operating conditions.

CONCLUSIONS

1. The alinement of the tunnels, Figure 2, is satisfactory for diver-
sion flows and spillway flows.

2. Preliminary tests on a 1:88 scale model indicated that the most
satisfactory invert angle for the flip buckets was 35°. Subsequent
tests on the 1:63.48 spillway model confirmed this.

3. A low curved concrete wall placed adjacent to the right canyon
wall will protect the canyon wall from further undermining and
erosion damage by diversion flows, Figures 24 and 25.

4. The spillway approach channels were greatly reduced from
their original size and still provided extremely smooth flow con-
ditions, Figures 26 through 36.

5. Flow through the crest sections was excellent and no adverse
pressure conditions were noticed, Figure 38. The maximum dis-
charge of 138,000-cubic feet per second (cfs) per tunnel was
obtained at reservoir elevation 3711, the value used for design
purposes, Figure 39.

6. The preliminary tunnel transition was too abrupt. A surface
fin formed in the center of the tunnel and pressures on the side-
walls were in the cavitation range, Figure 38.

7. The longer recommended transition, Figure 42, is adequately streamlined and provides smooth flow conditions with no adverse pressures on the sidewalls, Figures 43 and 44.

8. Flow in the 41-foot-diameter tunnels was excellent at all discharges.

9. The preliminary downstream circular-to-rectangular tunnel transition was too short, as indicated by severely subatmospheric pressures in the lower corners, Figure 45. Increasing the transition length from 70 to 100 feet increased the pressures to a satisfactory degree. This transition was eliminated in the recommended design.

10. The preliminary flip buckets, which were rectangular in cross section, were replaced by a bucket in which the circular invert of the tunnel intersected the vertical curve of the bucket, Figures 60 to 63. This type of bucket also eliminated the need for the circular-to-rectangular transition.

11. The flip buckets were moved upstream to the tunnel portals, eliminating about 200 feet of open channel.

12. The outside walls of the buckets were turned inward 7 feet to direct the flow in a more favorable pattern at their impact points, Figures 65 and 66.

13. The outside wall of the left bucket was extended 32.5 feet downstream from the lip to deflect the flow from the canyon wall.

14. Pressure measurements on the wall and invert of the left bucket showed that pressures as great as 211 feet of water should be considered in the structural design of the bucket, Figure 64.

15. The river outlets were arranged to distribute the jets in a fan shape, reduce their erosive tendencies in the river channel, and lessen the amount of riverbed material that had been carried upstream into the powerplant afterbay, Figures 71 and 72.

16. Erosion tests indicated that 24- to 30-inch-diameter riprap would be adequate to protect the powerplant tailrace channel.

17. Tailwater drawdown curves indicated that the tailwater elevation at the powerhouse will be as much as 30 feet lower than the downstream tailwater elevation for the maximum spillway discharge of 276,000 cfs, Figure 76.

18. Uplift pressures on the tailrace concrete slab were found to be 3 feet of water or less, Figure 77.

2

ACKNOWLEDGMENT

The final plans evolved from this study were developed through the cooperation of the staff of the Concrete Dams Section of the Dams Branch, Division of Design.

INTRODUCTION

Glen Canyon Dam is the principal feature of the Colorado River Storage Project. It is located on the Colorado River in north-central Arizona approximately 15 miles upstream from Lee's Ferry and 13 miles south of the Utah border, Figure 1. The dam is a concrete arch structure 710 feet high and approximately 1,550 feet long, Figure 2. The reservoir, at normal water surface elevation 3700, will have a surface area of 161,400 acres and a capacity of 27 million acre-feet, Figure 3. It will extend 186 miles up the Colorado River and 71 miles up the San Juan River. The reservoir will be used for regulatory storage and for the development of power in the eight-unit 900,000-kilowatt (kw) powerplant.

The principal hydraulic features of the dam are two tunnel spillways and the river outlet works. The tunnel spillways are located in each abutment. The spillway entrances are located about 600 feet upstream from the dam and consist of unlined approach channels and reinforced-concrete crest structures, Figures 2 and 3. Flow in each spillway is controlled by two 40- by 52.5-foot radial gates. Downstream from each intake structure is a transition from a flat-arch roof section, 89 feet wide by 52 feet high, to a circular section 482.5 feet in diameter, Figure 4. A tapered circular transition reduces the tunnel diameter to 41 feet in 180 feet. The remainder of each tunnel is 41 feet in diameter, terminating in flip buckets at river level. The capacity of each spillway is 138,000 second-feet.

The studies on the spillways included investigation of flow conditions in the approach channels, gate structure, tunnel transitions, tunnels, flip buckets, and downstream river channel. Studies were also conducted to determine tunnel alinement and flow conditions at the downstream tunnel portals during diversion.

The river outlets are located downstream of the dam near the left abutment, Figure 2. Flow through the outlets is controlled by four 96-inch hollow-jet valves placed above the maximum tailwater to discharge horizontally into the atmosphere. Investigations of the river outlets were limited to alinement of valves and flow characteristics in the river when operating singly, jointly with the spillways, or with flow through the powerplant.

3

THE MODELS

Two models were used in the tests on the spillways and outlet works. The first was a 1:88 scale model for preliminary investigations of the diversion tunnels and flip buckets, Figure 5. The second was a comprehensive 1:63.48 scale model of the spillways and outlet works, Figure 6.

The 1:88 scale model was contained in a 12- by 30-foot box and included an equivalent of approximately 1,000-foot lengths of the horizontal portion of each diversion tunnel and a sufficient area of the canyon and river channel in the vicinity of the tunnel portals so that exit flow conditions during river diversion could be evaluated, Figure 5. Water was supplied to each tunnel through separate pumps and measured by laboratory orifice-venturi meters in each supply pipe. Computed flow depths and velocities in the tunnels were established by slide gates placed at the upstream end of each tunnel section. The river water surface level was regulated by a tailgate at the downstream end of the model, and water surface elevations were obtained by means of point gages placed at appropriate locations.

The flip buckets used in the preliminary investigations were constructed of concrete screeded to sheet metal templates.

The 1:63.48 scale model covered a floor space of approximately 27 by 90 feet. The headbox containing the portion of the model upstream from the dam was 14 feet high, and the tailbox containing the downstream river channel was 3 feet high, Figure 6. Incorporated in the model were a 1,000-foot reach of the canyon upstream from the dam and about 3,500 feet of river channel downstream from the dam.

The vertical drop in the model tunnels was 1.35 feet (model) greater than the scaled prototype dimension, and the lengths of the horizontal tunnel sections were reduced by 5 feet (model) to compensate for the added friction loss in the model. These adjustments to the tunnel lengths and fall assured that the flow velocity at the elbows and portals were correctly represented. Tables 1 and 2 show the friction loss computations for maximum discharge through the model and prototype.

The spillway crest sections, the excavated approach channels between the canyon edge and the spillways, and the flip buckets at the downstream end of the tunnels were modeled in smooth concrete screeded to sheet metal templates. The crest piers were

4

constructed from wood and the radial gates were made from galvanized sheet metal. The topographic features of the canyon walls and the outline of the arch dam were modeled in concrete placed over wood templates and expanded metal lath. The river channel in the canyon downstream from the dam was represented with a movable sand bed. The general exterior outline of the powerhouse was constructed from waterproofed plywood. Powerplant flows in the river channel were accurately represented by an independent water supply. The river outlet valves were machined from brass stock and were individually operated. The tunnel transitions at the spillway portals and the tapered tunnels downstream of the transitions were made in clear plastic formed under heat over wood patterns or molds. The 41-foot-diameter tunnels were represented by extruded plastic pipe. The nominal diameter of this pipe was 8 inches but the actual inside diameter was 7-3/4 inches which determined the model scale.

Water was supplied to the model from the central laboratory supply system and measured by venturi meters. The maximum combined spillway discharge of 276,000 cfs was represented by a model discharge of 8.6 cfs. Water surface elevations in the reservoir were determined from a point gage in a stilling well. The opening to the stilling well was in the center of the headbox, well upstream from the effects of the spillway backwater curve. Tailwater levels were controlled by an adjustable tailgate at the downstream end of the tailbox. Tailwater elevations were measured on staff gages located at the end of the tailbox and on the face of the powerhouse. Pressures on the spillway crest, transition, tunnels, and flip buckets were determined from piezometers connected to open-tube manometers. Care was taken to make these piezometer openings normal and flush with the flow surface, burr free, and without change in direction for a distance of at least 3 diameters from the surface.

THE INVESTIGATION

Diversion Studies

During construction of the dam, the entire riverflow was diverted through two 41-foot-diameter concrete-lined tunnels, one on each side of the river channel, Figure 7. When the tunnels are no longer needed for river diversion, about 1,000 feet of the downstream sections of both tunnels will become a part of the tunnel spillways.

In the initial design planning, the diversion tunnels were 50 feet in diameter, unlined, and approximately 2,500 feet in length. Tests performed to determine whether the sandstone through which the tunnels were bored could withstand the erosive force

of sediment-laden, high-velocity flow indicated that the diversion tunnels should be lined. 1/ Accordingly, lined tunnels were specified and the diameter was reduced to 44 feet. Subsequent to the diversion studies, the diameter of the tunnels was further reduced to 41 feet to match the final size requirement for spillway discharges.

Hydraulic model studies were requested to check the alinement and elevation of the diversion tunnels with respect to the river channel. This model also was used for preliminary investigations of the flip buckets at the end of the tunnel spillway.

The two discharge quantities used for the diversion studies were 30,000- and 65,000-cfs per tunnel. Tests were made with the right tunnel operating singly and with both tunnels operating. Since the intake portal of the left tunnel is about 35 feet higher than the right tunnel intake, the left tunnel will not operate singly.

Two tailwater elevation curves were used during the studies, Figure 8. One was the "Phoenix" curve, derived on the assumption that Marble Canyon Dam will be built downstream from Glen Canyon Dam and will control the tailwater elevations. The second curve was based on observed water surface elevations for the extreme lower range of flows at Glen Canyon and extended for the higher flows from the shape of the Lee's Ferry tailwater curve which is based on a comprehensive range of discharge measurements. (Lee's Ferry is a permanent gaging station downstream from Glen Canyon.) The tailwater curves, shown on Figure 8, indicate a difference in elevation of approximately 5 feet for low flows and 17 feet for maximum flows; the Marble Canyon curve shows the higher tailwater elevations.

The investigations showed that, in general, the tunnel alinement and grade were satisfactory for diversion flows. The curved exit wall downstream from the right tunnel caused some eddies in that vicinity, but when the curved wall was replaced by a straight wall the eddies were eliminated and the flow was entirely satisfactory. Figures 9 and 10 show the flow conditions in the river channel during diversion.

Preliminary Flip Bucket Studies

Tunnel alinement. --For these studies, no changes were made in the 1:88 scale model other than to install the flip buckets at the tunnel portals. One purpose of the investigations was to determine

1/Report Hyd-423, Erosion Studies on Sandstone Through Which the Glen Canyon Dam Diversion Tunnels Will Pass. Glen Canyon Dam, Colorado River Storage Project.

if the alinement of the tunnels would be satisfactory when the higher velocity spillway flows were directed into the downstream river channel. The principal objective of the tests, however, was to determine the optimum angle of flip for the invert of the buckets. The flip angle was evaluated on the basis of water surface drawdown at the powerplant tailrace, wave action in the river channel, and the general appearance of the jets leaving the buckets. A maximum discharge of 142,000-cfs per tunnel for one- and two-tunnel operation with both tailwater regimens was used for these tests. The 142,000-cfs maximum discharge was reduced to 138,000 cfs before the 1:63.48 scale model studies were started.

Five buckets were investigated, Figure 11. The buckets differed in the angle of the flip which was accomplished by varying the length and radius of the invert curve. The location and elevation of the bucket lip were the same for all buckets.

The tests showed that the alinement of both tunnels was satisfactory for all spillway flows. The elevation of the bucket lip also appeared satisfactory for the lower tailwater conditions; with the higher tail-water conditions, the water surface touched the lower nappe surface, causing the jet to intermittently depress. However, since the tailwater curves are tentative, it was decided that the bucket lip elevation should not be changed until a final tailwater curve had been determined.

Water surface drawdown. --The following test procedure was used in determining the water surface drawdown. With either tunnel operating alone or with both tunnels operating simultaneously, the tailwater elevation was set at the point gage located approximately 500 feet downstream from the tunnel portals; after allowing adequate time to insure that the flow in the river channel had become constant, the water surface elevation at the approximate location of the powerhouse tailrace upstream from the tunnel was determined. The difference in water surface elevation between the two stations was used as a measure of the drawdown.

The curves on Figure 12 show the water surface drawdown for the different flip buckets. The curves indicate that for single-tunnel operation, the greatest drawdown occurred with the 30° flip bucket and the least drawdown with the 35° bucket. Due to the relative position of the buckets and their alinement with the river channel, the drawdown for the 30° and 35° buckets was greater when the left tunnel was operating than when the right tunnel was operating. For the 25°, 40°, and 45° buckets, the drawdown was about the same when either tunnel was operating. During single-tunnel operation, there was greater water surface drawdown with the higher tailwater elevation.

7

When both tunnels were operating, the least drawdown occurred with the 40° and 45° flip buckets using the high tailwater and with the 35° flip bucket using the low tailwater. The greatest drawdown occurred with the 25° bucket using the high tailwater, and with the 30° bucket using the low tailwater.

On the basis of the drawdown measurements and the general flow appearance, it was decided to use the 35° flip curve for both tunnel spillways. Figure 13 shows the operation of three of the buckets during maximum discharge and the Marble Canyon (high) tailwater elevations.

Diversion Studies in 1:63.48 Model

Prototype operation. --The hydraulic model investigations were performed concurrently with the construction of the dam. During the first 2 years of construction, most of the riverflow had been diverted through the right diversion tunnel; only small quantities had passed through the left tunnel. The diversion flows caused some undercutting of the right canyon wall and the appearance of the flow downstream from the tunnel portal indicated that erosion of the river channel was taking place, Figure 14A.

Model studies were initiated to investigate methods proposed to prevent further erosion damage. In the early phases of the diversion model studies, the full extent of the prototype erosion became apparent when about 20,000 cubic yards of the canyon wall immediately downstream from the portal fell into the river, Figure 14B. The rockfall had little effect on the diversion flow; the headwater rose for a day or two, but returned to its former elevation after the small debris had washed out. However, in order to forestall further slippage of the canyon wall, the decision was made to close the diversion gates of the right tunnel and to make repairs to the area where the rock wall had slipped. These repairs would include stripping of the canyon wall to prevent further falling of the rock, construction of a protective concrete wall along the right canyon wall, and filling the eroded hole in the channel with concrete to elevation 3130.0. While these repairs were underway, diversion flows were passed through the left tunnel.

Initial studies. --One proposal for the emergency repair of the scour hole was to install the bottom half of the flip bucket; at a later date the top portion of the bucket would be added for final operation. However, this two-stage bucket had been tested during left diversion tunnel studies as a probable solution for difficulties with the diversion water striking the canyon walls and had operated so poorly that the plan was abandoned. The results of these studies are given in Hydraulics Branch Report No. Hyd-468.

8

The first emergency repair method investigated in the 1:63.48 model was to install the complete flip bucket at the tunnel portal. For most diversion discharges, the water poured over the bucket lip and side without springing clear of the bucket. These flow conditions could possibly cause additional rock erosion around the completed bucket; therefore, the studies were directed toward developing adequate protection to the canyon wall and channel downstream from the portal.

Side channel spillway.--The downstream cofferdam is located adjacent to the bucket. To prevent the diversion flows from eroding the cofferdam, it was proposed to construct a concrete-lined channel parallel with the flip bucket so that water passing over the side of the bucket would be carried downstream away from the cofferdam. Two types of channels were proposed; one was a deep channel on a flat slope, and the other was a comparatively shallow channel on a steep slope. Since it was desirable to keep the amount of rock excavation involved to a minimum, the shallow channel was first tested in the model, Figure 15.

Flows up to 35,000 cfs did not overtop the sidewall of the channel, although a small amount of splashing did wet the adjacent cofferdam, Figure 16. Flows above 35,000 cfs overtopped the wall and seriously damaged the adjacent cofferdam, Figure 16. On the basis of these tests, further modifications were needed to prevent damage to the cofferdam.

The side channel spillway was redesigned so that it was wider and the left wall was curved toward the river. A 5-foot-wide seawall or coping strip was placed on top of the wall to deflect the flow downward, Figure 17. This protective device performed exceptionally well and protected the cofferdam against damage for flows up to 100,000 cfs, Figure 16. In addition to directing the flow away from the cofferdam, the jet spread into the river channel and relieved some of the pressure against the right canyon wall.

Although this structure provided the necessary protection, a design analysis showed that it would be too expensive for a temporary structure because of the difficult construction and the lack of good rock foundation in the area.

Deflector walls.--The rock fall from the canyon wall, described previously, required that protection be provided to prevent further undercutting and slippage of the canyon wall. It was decided, therefore, to fill any eroded holes in the channel floor with concrete and to develop a protective wall that would deflect the flow away from the canyon wall.

9

Cross sections received from the field indicated that the canyon wall had been undercut as much as 50 feet to the right of the projected right side of the tunnel. The model canyon wall was modified to represent this undercutting and the overhanging rock above the undercut section was stripped back, Figure 18.

To evaluate this proposal in the model, the right canyon wall and the channel floor downstream from the tunnel portal were remolded in a weak, easily erodible, sand-cement mixture. The erodible mixture was composed of aluminous cement which formed a concrete that attained its ultimate strength in 24 hours. The strength of the mixture was such that it would begin to erode at a model velocity of about 1.5 feet per second (fps). 2/ The cement-sand ratio of the mixture was 1:70, by weight, the water-sand ratio was 1:5, by weight. The procedure for placing the material was to mix the three ingredients thoroughly, and to tamp the mixture firmly in the model with a wood hand trowel. The mortar was placed to a depth of about 3 inches on the floor and 4 to 12 inches on the sidewall. The mixture was allowed to cure for a minimum of 24 hours before an erosion test was started.

A vertical deflector wall was the first protective device investigated with this model arrangement. The wall was placed to the right of the tunnel and extended 50 feet downstream from the tunnel portal and converged about 8 feet toward the tunnel centerline, Figure 18. This wall would eventually serve as a backing for the right side of the permanent bucket.

Tests showed that this deflector wall failed to direct the water into the river and was ineffective in preventing undercutting. The flow impinged on the eroded area and tended to increase the canyon wall erosion downstream from the presently eroded area, Figure 18.

The deflector wall was revised by increasing the amount of deflection at the end of the wall and by superelevating the floor, Figure 19. This revised wall accomplished the purpose of deflecting the water away from the canyon wall, Figure 19.

Although the model studies indicated that the superelevated deflector wall was satisfactory under the assumed eroded conditions, it was decided that, since the true shape and depth of channel bed erosion downstream from the tunnel portal were not known, an alternate scheme should be developed for use. The choice of the schemes would be made after the area was unwatered and the extent of erosion was determined.

2/ Appendix C, Fontana Project, Hydraulic Model Studies, Technical Monograph No. 68, Tennessee Valley Authority.

The alternate scheme consisted of a low wall laid against the canyon wall along the curve of the eroded portion and extending downstream from the tunnel portal. A structure of this type was preferable to the superelevated structure because its removal after temporary use during the diversion period was unnecessary.

The first wall investigated was 250 feet long and 30 feet high and extended in a long-radius curve from the tunnel portal to the point where the canyon wall breaks away from the right flowline, Figure 20. In general, this wall was satisfactory for all flows, Figure 21. However, for discharges between 35,000 and 50,000 cfs at certain tailwater elevations, an unstable flow condition developed and caused the water over the entire width of the river to fluctuate in a harmonic motion. This phenomenon was caused by the tailwater alternately submerging the flow and being swept away by the high velocity flow from the tunnel. The resulting surges were about 15 feet high with a period of about 30 seconds. Action of this type would probably cause extensive damage to the cofferdam. Figure 22 compares the flow conditions when the jet emerging from the tunnel is partially submerged by the high point of the tailwater surge and when the jet flows free during the low stage stage of the tailwater surge. In an attempt to eliminate the surging action, the curved wall was replaced by a straight wall that extended in a direct line from the right side of the tunnel portal to the downstream end of the curved wall, Figure 23. Surges as large as those observed with the curved wall still persisted in the discharge range between 35,000 and 50,000 cfs.

Since the straight wall did not improve the flow conditions and the curved wall would require about 50 percent less concrete to construct, testing was continued using the curved wall. The surging action in the river was caused by the flow from the tunnel sweeping the water away from its path; this displaced water moved across the river in a surge, impinged on the left bank and was deflected back across the river toward the right bank where it again impinged on the flow emerging from the tunnel. It was reasoned that if the flow in the river could be kept from impinging on the tunnel flow, the surging action would not start. Further testing showed that either a spur dike or wall placed about 150 feet from the canyon wall with its long axis parallel to the diversion tunnel centerline and extending about 150 feet downstream from the existing cofferdam prevented the unstable flow and resulted in satisfactory operation for the entire range of discharges.

To effect the repairs at the tunnel portal and canyon wall, this area must be isolated from the river by a cofferdam so that it can be pumped dry. The cofferdam will extend between the existing main river cofferdam and the canyon wall about 300 feet downstream from the tunnel portal. It was recommended that the spur

dike be the remains of this cofferdam. In other words, only that portion of the cofferdam near the canyon wall would be removed to allow passage of the diversion flow and the remainder would serve as the spur dike between the tunnel flow and the river channel. Model tests of this scheme indicated that the dike was satisfactory and fairly stable but might require some riprap protection on the nose of the dike.

Instead of the curved wall, a wall consisting of three chords was used, Figure 24. This wall was found to be less effective than the curved wall but minor differences in performance were justified by the lower construction costs. It was demonstrated by constructing and later removing the downstream part of the cofferdam that the dike thus formed would be effective in preventing the harmonic motion surges in the tailrace for discharges in the 35,000- to 50,000-cfs range.

The wall consisting of the three chords, Figure 25 and the spur dike formed from the cofferdam were installed in the prototype. Subsequent operation of the diversion tunnel showed this scheme to be very effective in handling the diversion flows.

Spillway Approach Channels

The portals or gate control sections of the tunnel spillways were located inland from the canyon rim to provide adequate rock cover for the tunnels and to obtain an exit angle into the river. Open cut approach channels extended from the canyon edge to the spillway portals to provide flow passages between the reservoir and the tunnel spillways. The channels were unlined and, in plan, were in the form of moderate curves. The sides of the channels were excavated with 1/4:1 side slopes and converged slightly to provide a gradual acceleration of the flow in the approach channel. The approach channels were studied to determine the minimum size and optimum alinement that would provide smooth flow conditions at the gate control sections and spillway portals.

Left Approach Channel

Preliminary channel. --The preliminary left approach channel, Figure 26, had a bottom width of about 400 feet at the canyon rim and gradually converged to approximately 110 feet wide at the spillway crest. In plan, the left side of the channel followed a mild reverse curve; the right side followed a large-radius curve.

Extremely smooth flow conditions throughout most of the channel at maximum discharge indicated that the width of the approach channel was more than adequate. The only disturbances were in the form of eddies and reverse flow currents along the right

boundary, Figure 27. Flow velocities were generally higher in the right side of the channel than in the left. At the channel entrance, the velocity was 7.3 fps near the right side, 4.3 fps at the center of the entrance, and 2.1 fps near the left side. At the spillway entrance, the velocity averaged about 15.5 fps throughout the flow section. The velocity distribution in the preliminary approach channel is tabulated on Figure 26.

These tests indicated that flow conditions in the approach channel and particularly at the spillway gate section were entirely satisfactory. They also suggested that satisfactory flow conditions possibly could be obtained by reducing the length and width of the channel. Testing of smaller approach channels, therefore, was continued.

First revision. --The channel width was reduced by moving the left wall in about 70 feet at the canyon rim and fairing it into the original wall about 100 feet upstream from the gate section. The right side of the channel was modified by using a short-radius curve at the canyon edge, thus providing a more curved and abrupt entrance, Figure 26.

Generally the flow in the revised channel was excellent. Flow disturbances along the left wall were negligible, Figure 28A. A comparatively large contraction occurred at the curved end of the right wall where the water surface was depressed 4 feet and eddy currents and reverse surface flow extended along the wall from the depressed water surface to the gate section. However, the effect of these eddy currents and reverse flow did not extend beyond the tunnel portal and the flow distribution in the portal transition was very good.

Second revision. --Since excellent flow conditions still existed in the approach channel, it appeared that the channel width could be further reduced without adversely affecting the flow conditions. Accordingly, the left wall at the canyon entrance was extended downstream an additional 50 feet and faired into the original boundary about 50 feet upstream from the spillway entrance, Figure 26. The right wall was further modified by using a longer radius curve at the canyon edge and a comparatively straight approach to the spillway entrance.

At the maximum discharge, the flow appearance in the channel was very good. A negligible rippling on the water surface near the left wall indicated that the reduction in channel width was near the optimum. The amount of contraction on the right side of the channel was still about 4 feet along the curve at the canyon entrance.

Eddy currents and reverse flow along the boundary were more prevalent, Figure 28B. However, these distrubances did not extend beyond the gate section and the flow appearance in the transition remained satisfactory.

Third revision. --The width of the approach channel was further reduced by moving the left wall an additional 20 feet downstream at the canyon rim; the right wall was not altered in this revision, Figure 26.

At the maximum discharge, standing waves emanated from the left wall and extended from the left wall toward the center of the channel in the direction of flow. These waves were less than 6 inches in height and caused no adverse flow conditions in the tunnel. The water surface drawdown and reverse flow eddies along the right wall were the same as those observed for the second revision. The appearance of the water surface in the channel at the maximum discharge is shown on Figure 29A.

Flow velocities in the channel were generally higher than the velocities in the preliminary channel due to the greatly reduced flow area. However, the velocities still tended to be higher on the right side than on the left. At the channel entrance, the velocities near the right boundary were about 11.0 and 8.0 fps at the center of the channel, and 6.1 fps near the left wall. Velocities immediately upstream from the gate section were comparatively uniform. The velocity distribution in the channel is tabulated in detail on Figure 26.

Fourth revision. --The only undesirable feature in the revised left channel was the flow appearance along the right wall. Although detracting in overall operating appearance, the water surface drawdown at the upstream end and the reverse flow eddy currents between the point of drawdown and the gate section did not affect the hydraulic characteristics of the entrance. The tests indicated that these conditions probably could be alleviated by modifying the right side of the approach channel. Therefore, the comparatively abrupt curvature of the right wall was cut back and replaced with a long-radius curved wall from the reservoir to the gate section. This change substantially reduced the length of the right sidewall, Figure 26.

The long-radius curve did not improve the flow conditions along the right wall. At the maximum discharge, a 4-foot drop in the water level still occurred near the middle of the curved wall with eddies and reverse flow currents between the depressed water surface and the gate section.

Recommended channel. --Although the long-radius curve did not change the flow conditions on the right side of the approach, the channel width at the canyon rim was considerably increased. Since previous tests had shown that a comparatively narrow approach channel was adequate, the width could be further reduced without upsetting the excellent flow conditions and a further reduction in the quantity of rock excavation would be accomplished. Therefore, the left wall was moved 20 feet downstream at the canyon edge and faired into the previously revised wall, Figure 30.

At the maximum discharge, the flow appearance in the channel was satisfactory, Figure 29B. The flow velocities at the channel entrance were more uniform and generally lower than the velocities observed in the third revision; the average velocities at the canyon rim were 6.7 fps near the right bank, 7.8 fps at the center, and 6.2 fps near the left bank. The average velocities immediately upstream from the gate section were 18.1 fps on the right side, 16.8 fps at the center, and 15.1 fps near the left bank. Although the flow velocities at the gate section were slightly less uniform than those observed in the preliminary or third revised channel, the velocity distribution was considered entirely satisfactory. Additional flow velocities in the approach channel are shown on Figure 31.

Right Approach Channel

Preliminary channel. --The arrangements of the preliminary right and left approach channels were similar. The gate section and tunnel portal of the right spillway were set a greater distance back from the canyon edge than the left spillway gate section. This difference permitted a more gradual or longer radius curve for the left wall of the right approach channel. The outside or right boundary of the right approach channel was in the form of a moderate "s" or reverse curve between the canyon edge and the gate section. The bottom width of the right channel was 460 feet at the canyon rim and reduced to about 110 feet wide at the gate section, Figure 32.

At the maximum discharge, the flow conditions in the approach channel were ideal. Flow along the right side was excellent except for small, very minor eddies at the channel entrance. The water surface was depressed a maximum of 2 feet along the left side where the curvature was greatest. Flow disturbances in the form of standing waves less than a foot high formed approximately parallel to the wall. The approaching flow piled up to a height of 1 or 2 feet in front of the piers on each side of the spillway entrance. This pileup was caused by a partial recovery of the velocity head of the flow striking the flat surface of the pier

15

face, Figure 33. Velocities at the channel entrance were about 3.9 fps near the right side, 3.7 fps in the center, and 4.5 fps near the left side. In front of the spillway entrance, the velocities were about 14.6 fps on the right side, 15.1 fps in the center, and 15.8 fps on the left side. The velocity distribution for the maximum discharge in the preliminary channel is shown on Figure 32.

Because generally excellent flow conditions and comparatively uniform velocity distribution existed in the preliminary approach channel, the tests were continued on approach channels requiring less excavation.

First revision. --The preliminary channel was modified by moving the right boundary downstream about 100 feet at the canyon rim and fairing it into the preliminary boundary about 100 feet upstream from the gate section, Figure 32. The left side of the channel was not modified.

At the maximum discharge, the appearance of the flow in the channel was very good, Figure 34; additional eddy currents developed at the upstream end of the channel near the right side, but these seemed to be caused by the shape of the natural topography rather than by the restricted flow passage. The flow pattern along the left side was essentially the same as that observed in the preliminary channel.

Observations using dye streams and floating confetti confirmed that flow conditions in the restricted approach channel were excellent and indicated that the channel width might be further reduced.

Second revision. --The right wall was moved 75 feet farther downstream at the entrance and faired into the preliminary wall similar to the first revision, Figure 32. No changes were made to the left side.

Again, the appearance of the flow in the channel was very good, Figure 34. Surface disturbances appeared in the center and along the left wall of the channel, indicating that the channel width was near the minimum required for satisfactory flow.

Third revision (recommended). --The right wall was moved downstream an additional 30 to 40 feet at the canyon edge and faired into the original wall similar to the previous changes, Figure 30. No changes were made in the left wall.

The standing waves and surface disturbances first noticed in the previous revision at maximum discharge were more apparent,

Figure 34. However, the flow conditions were considered satis-factory since the water surface at the gate section was symmetrical without excessive disturbances.

Flow velocities in the channel near the canyon rim were about 6.2 fps near the right side, 8.5 fps in the center, and 10.6 fps on the left side. Immediately upstream from the gate section, the velocities were about 14.2 fps on the right side, 13.2 fps in the center and 16.9 fps on the left side. Additional flow velocities in the approach channel are shown on Figure 31.

Fourth revision. --A comparison of the above flow velocities with those recorded for the preliminary channel indicates that a narrow approach caused a slight flow concentration on the left side of the approach channel. To alleviate this asymmetrical velocity distribu-tion and still retain the narrow approach channel, a small fill was placed in the upstream portion of the approach channel. The fill near the canyon rim sloped laterally from a height of 15 feet at the left bank to the original floor elevation on the right bank. The fill also sloped downward in the direction of flow to the original floor elevation about 120 feet upstream from the gate section.

The raised floor caused only a slight redistribution of the flow at the smaller discharges. At the maximum discharge, there was no noticeable difference in the flow except near the gate section where more surface disturbances in the form of waves and eddies were observed, Figure 36. These disturbances carried down-stream into the tunnel transition and caused a rough water surface and uneven flow distribution in the tunnel. Because of these undesirable flow conditions, the narrow approach channel with a horizontal floor (third revision) was chosen for prototype use.

The recommended approach channels were considerably shorter and narrower than those proposed in the preliminary plans. It was estimated that the reduction in the volume of rock excavation was about 440,000 cubic yards.

Spillway Crest (Overflow Section)

The right and left spillways are identical from the gate section to the horizontal tunnel. Each overflow section includes two symmet-rical 40-foot-wide flow passages separated by a center pier, Fig-ure 37. Flow is controlled by two 40- by 52.5-foot radial gates. The ogee section in each passage is turned inward 6° to provide converging sidewalls and the center pier is tapered to provide a constant width of passage through the ogee section. The stream-lined nose of the center pier and the side piers extend upstream from the ogee section to assist in developing good flow condition in the control section. The radial gates seat 11 feet downstream

17

from the crest axis at an elevation 6 inches below the crest elevation.

Since the two spillways are identical, certain model data including water surface profiles, piezometric pressures, and general flow characteristics in the tunnels and transitions were obtained only in the left spillway, but apply equally well to the right spillway. Although the two approach channels were slightly different, the flow appearance and velocity distribution at the gate sections indicated that the flow conditions in the two structures were similar.

Crest pressures. --Piezometers were placed in the overflow section along the centerline of the left bay of the left spillway. Since flow conditions were similar in the four bays of the two spillways, piezometers were not placed in the other bays. Pressure measurements made for free flow at the maximum discharge showed no subatmospheric pressures on the crest profile. The piezometer locations and pressures at each piezometer are shown on Figure 38. Pressures for gate controlled discharges were either near or above atmospheric, Figure 44.

Discharge capacity. --The discharge capacity of both spillways for controlled and free flow was determined from the model. The flow quantities were obtained with both spillways operating; for controlled flows, all four gates were equally opened. Several scattered points were obtained with only the left spillway operating to determine if the flow through both spillways would be equal. At the points checked, the flow was exactly 50 percent of the quantity that had been measured for similar reservoir elevations and with both spillways operating.

The discharge capacity of one gate for free flow and for controlled flow at gate openings in 5-foot increments is shown on Figure 39. At the maximum design discharge of 138,000-cfs per spillway, the reservoir elevation was 3710.65. The discharge coefficient for the maximum flow was 3.48.

Tunnel Spillway Transition

Preliminary. --The change in cross section from the rectangular spillway crest section to the 41-foot-diameter inclined tunnel was accomplished by a curved transition from the rectangular spillway to a 50-foot-diameter circular tunnel followed by a section of tunnel tapering from 50- to 41-foot diameter, Figure 40. The horizontal projected length of the transition invert was about 101.4 feet with a vertical drop of 94.6 feet. The horizontal length of the tapered tunnel was 135.9 feet with a vertical drop of 194.1 feet.

18

In side elevation, the transition invert, spring lines of the upper and lower radii, and the crown of the transition were parabolic curves as shown on Figure 40. In plan, the sides converged in a straight line.

The invert of the tapered tunnel sloped downward at an angle of 55°. The top and sides of the tunnel converged lineally until the 50-foot diameter was reduced to 41 feet.

The center pier on the crest extended down into the transition section for a horizontal distance of 65 feet. The downstream end of the pier rose vertically from the invert for 35.24 feet then extended to the roof on a line normal to the roof. The pier tapered from 8.5 feet wide at the start of the transition to 5.0 feet wide at the end. The nose of the pier was stream-lined, in plan, with a 15-foot radius and the downstream end of the pier with a 2.5-foot radius.

Flow conditions in the preliminary transition were unsatisfactory. At the small discharges, up to about 50,000 cfs, a fin formed in the tunnel which, although not pleasing to the eye, caused no apparent difficulty. For flows between 50,000 and 100,000 cfs the flow exhibited some instability downstream from the transition; however, the center fin had reduced in magnitude. For discharges greater than 100,000 cfs, the flow instability increased considerably; a definite "hump" formed in the water surface near the top of the tapered tunnel section; and the flow appeared to separate from the sidewalls, Figure 41. These observations indicated that the change in section was too abrupt.

Piezometers were installed throughout the walls and invert of the transition section, Figure 38. Pressure readings at piezometers located in the upstream end of the tapered tunnel indicated pressures in the cavitation range at the maximum discharge. Other piezometers in the sidewalls and curved corners of the transition showed subatmospheric readings during maximum discharge conditions; piezometers on the invert indicated above atmospheric pressures for all discharges. A complete tabulation of the pressures is shown on Figure 38.

Recommended transition. --Data from pressure measurements, water surface profiles, and general flow appearance were analyzed to determine what modifications should be made to the transition section to provide satisfactory operation. It was concluded that a curved transition approximately 50 percent longer than the preliminary would provide sufficient stream-lining to insure stable flow conditions. In addition, it was

19

reasoned that the pressures on the sidewalls of the tapered tunnel would be improved if the convergence was accomplished with curved sidewalls tangent to the tapered tunnel rather than with straight sidewalls and an angular intersection with the tapered tunnel.

The recommended transition, Figure 42, had the same general appearance as the preliminary except that the side convergence was accomplished in a curve and the horizontal length was increased about 26 feet.

The flow stability in the modified transition was greatly improved. The general appearance of the water surface in the tunnel was not improved; the center fin that formed downstream from the center pier was still present at low discharges, but did not impinge on the roof or cause unsymmetrical flow in the tunnel. The fin was not present at flows greater than 75,000 cfs. At discharges greater than 75,000 cfs the water surface drawdown at the side and center piers at the tunnel portal caused surface disturbances that carried down into the transition; this rough water surface did not create unsatisfactory flow conditions in the tunnel, Figure 43.

Piezometers were installed on the invert and sidewalls of the left tunnel transition in locations similar to the piezometers in the preliminary transition. The pressures obtained on the invert were the same as those observed in the preliminary transition. All pressures on the sidewalls where cavitation pressures had been observed previously were near atmospheric. A complete tabulation of the pressures is shown on Figure 44.

Since no pressures near the cavitation range were observed in this transition, and since the flow appearance was satisfactory, the transition was chosen for prototype installation.

Forty-one-foot-diameter Tunnels

Downstream from the tapered section, the tunnel is 41 feet in diameter and follows the 55° slope for about 75 feet and then changes direction to the near horizontal tunnel with a 350-foot radius bend. The near horizontal tunnels continue for approximately 1,080 feet for the left tunnel and 910 feet for the right tunnel, before emerging from the canyon wall, Figure 42.

Flow in the 41-foot-diameter tunnels was excellent at all discharges. The minor water surface roughness that was noticeable in the transitions and tapered sections had smoothed out in the first few feet of the constant diameter tunnel and continued smoothly through the vertical bend; consequently the flow in the horizontal tunnel was also satisfactory.

20

Piezometers were installed on the tunnel invert at intervals from the downstream end of the tapered section through the vertical bend. No subatmospheric pressures were indicated at any of the piezometers. The piezometers along the vertical bend showed the increased pressure due to the centrifugal force of the flow in the elbow; the maximum observed pressure in the elbow was equivalent to 88.7 feet of water, approximately twice the hydrostatic pressure.

Downstream Portal Transition

A transition was planned for the downstream end of each tunnel to guide the flow from the circular conduit to the rectangular channel between the tunnel portal and the flip bucket. Details of the preliminary transition, which was 70 feet long, are shown in Figure 45A.

The flow appearance in the transition was very good. However, piezometers along the lower corner of the transition indicated subatmospheric pressures in the cavitation range. Piezometers were installed as shown in Figure 45A. The piezometers along the bottom tangent line indicated above atmospheric pressures for the full length. The piezometers along the side tangent line showed above atmospheric pressures at the upstream end of the transition, 4 feet of water below atmospheric about 17 feet downstream from the start, and a steep increase to 13 feet of water above atmospheric at the third piezometer 11 feet farther downstream. The center row of piezometers indicated severe subatmospheric for the first 17 feet, the lowest pressure being 23 feet of water below atmospheric 3.5 feet downstream from the start of the transition. The pressure 29 feet downstream from the start of the transition increased to about 26 feet above atmospheric; pressures remained above atmospheric through the remainder of the transition, Figure 45A.

The subatmospheric pressures indicated that the change in cross section in the transition was too abrupt. Accordingly, the transition was modified so that it was 100 feet long with the centers of the radii on each side tracing a parabola, Figure 45B.

The appearance of the flow in the modified transition was excellent. Pressure readings from piezometers in locations similar to those in the preliminary transition were above atmospheric along the full length, Figure 45B. On the basis of these tests, the second transition was considered satisfactory for the field installation. However, during subsequent model investigations of the flip buckets downstream from the transition, it was determined that better bucket performance could be obtained if the semicircular invert of the tunnel was continued downstream and allowed to intercept the upward curve of the flip bucket. This not only provided good

bucket performance, but eliminated the expensive formwork needed
for the transition construction. The description of these investiga-
tions is included in the following section.

Flip Bucket Investigations

Preliminary.--In the preliminary layout, 41-foot-wide open chan-
nels extended downstream from the transitions terminating in flip
buckets. The combined length of the open channel and flip bucket
was 251.5 feet in the right tunnel and 280 feet in the left tunnel,
Figures 40 and 46. The bottom slopes of the channels were the
same as the circular tunnels. The inverts of the flip buckets con-
sisted of segments of a 109.92-foot radius circle. The flip angle
of both buckets was 35° above the channel floor, or approximately
35°-12' above the horizontal, Figure 40.

Because of the difference in their alinements and lengths of the
horizontal tunnels and open channels upstream from the buckets,
the left bucket was 319.64 feet farther downstream than the right
bucket. This bucket arrangement was very desirable hydraulically
because it spaced the spillway flow over a long reach of the river
channel and prevented a concentration of the jets in a relatively
small impact area.

The appearance of the flow from the left bucket was very good at all
discharges. The jet cleared the flip bucket smoothly with no appre-
ciable lateral spreading. However, the jet spread longitudinally
and the length of its impact area was comparatively long, particu-
larly for flows less than 50,000 cfs. Figure 46 shows the flow from
the flip bucket for two discharges, which were representive of the
full range of discharges.

The flow from the right bucket was also very good; the jet appeared
similar to the jet from the left bucket, Figure 47. However, the
alinement of the right tunnel was almost parallel to the canyon wall,
and for spillway flows of 75,000 cfs and larger, the right side of
the jet impinged on the canyon wall.

When both spillways were operating with a combined discharge of
150,000 cfs or less, the flow conditions were completely satisfac-
tory. For combined discharges greater than 150,000 cfs, the
conditions were fair. During small discharges, the jet impact
areas were independent of each other and the quantities involved
were so small in comparison to the size of the river channel
that no adverse flow conditions were noticed. During the larger
discharges, the jet from the left bucket landed near the center of
the river, well downstream from the structures; the jet from the
right bucket landed near the right side of the river with part of
the jet impinging on the canyon wall. The flow pattern resulted

in a concentration of flow along the right side of the canyon. Figure 48. This flow distribution caused an eddy current to originate near the left side of the left jet and to move upstream under the jet toward the impact area of the right jet. The eddy current carried some of the riverbed material that was being churned up by the force of the jets landing in the river and was deposited in a sandbar that extended across the river approximately in a line between the two buckets. The sandbar did not affect the spillway flow in the river but with no flow through the spillways and only the powerhouse in operation, the sandbar caused a 4-foot increase in the water surface elevation in the powerplant afterbay.

First revision. --Before tests were made to determine the positions of the buckets to obtain proper jet dispersion in the river, it was decided to eliminate the transition between the circular tunnel and the rectangular open channel by extending the circular tunnel until it intersected the vertical curve of the flip bucket, Figure 49. Figure 50 shows the revised left channel and flip bucket.

With this arrangement, the flow seemed to diverge at the lip of the bucket and resulted in considerably more lateral dispersion of the jet. At the maximum discharge, the jet covered the entire left half of the channel at the point of impact, Figure 50. This lateral dispersion of the jet eliminated the eddy that formed with the preliminary bucket and prevented the upstream sandbar deposit. However, a wide, high sandbar formed downstream from the impact area.

Second revision. --Based on the overall good appearance of the flow, coupled with the apparent cost advantages of eliminating the transition, the designers decided that the bucket with the circular invert in the channel should be used for both spillways. In addition, the location of the buckets was changed, so that they would be more nearly opposite one another, by moving the left bucket 100 feet upstream and the right bucket 100 feet downstream.

At the maximum discharge, the two jets landed in practically the same impact area. However, the jets from both buckets impinged on the canyon walls to a greater extent than previously. There was extensive erosion of the riverbed but all of the disturbed material moved downstream. The water surface was much rougher than it had been with the preliminary buckets. Figures 51 and 52 show the flow appearance from the buckets.

Third revision. --For the third revision, both buckets were moved upstream so that their vertical curve started 17.72 feet downstream from the tunnel portal. This arrangement resulted in a staggered impact area similar to that with the preliminary buckets.

The buckets were next moved upstream to the tunnel portals when subsurface exploration and field core drilling showed that the rock foundation on both sides of the canyon downstream from the tunnel portals was not as sound as expected.

In addition, the wall on the canyon side of each bucket was turned inward (toward the flow) 8 feet in a distance of 40 feet, Figure 53, and the opposite wall of each bucket was turned outward 4 feet in 20 feet.

With this revision, the jets landed in tandem and the deflection of the outer side of the jets was moderate for all discharges, Figure 56. The wall deflector caused the outside of the jet to rise vertically and fold over into the main body of the flow. The side of the jet next to the river was ragged and dispersed. The jets became more compact as the discharge increased. Figures 54, 55, and 56 show the flow conditions for several discharges representing the complete range of operation.

At the maximum discharge, surges in the river channel developed and caused a 6- to 8-foot variation in the water level in the power-house afterbay. This condition seemed to originate when a wave caused by the impact of the right jet moved diagonally upstream toward the left bucket, passed under the left jet, and reached the end of the bucket. The wave then rose to bucket lip and struck the lower surface of the jet causing the jet to depress. The depressed jet created another wave that moved toward the right bucket where a similar action would take place. This alternating action continued with the waves becoming progressively larger and eventually extending upstream into the powerhouse afterbay. Occasionally, an irregularity in the periodicity of the action would cause it to stop for short intervals. Although this action would have to be corrected before a bucket would be acceptable, it was decided to proceed with the tests to develop the wall deflectors.

Fourth revision. -- Before making major revisions of the flip buckets, several quick tests were made with several wall deflectors. These included deflectors having a width of 4 feet in the left bucket and widths of 6 and 7 feet in the right bucket. The diverging walls in the buckets were not changed.

Based on the results of these tests, the fourth revised buckets were developed. The wall deflector of the right bucket was maintained at 7 feet wide by 40 feet long; the diverging wall remained at 4 by 20 feet. In the left bucket, the 4- by 40-foot converging wall was extended downstream an additional 30 feet making a deflector 7 feet wide and 70 feet long; the 4- by 20-foot diverging wall was unchanged. Figure 57 shows the revised buckets.

The converging left wall of the left bucket had been extended an additional 30 feet downstream as a result of model studies on the tunnel plug outlet works, 3/ which were being conducted simultaneously with this study. During initial construction, the riverflow will be passed through the diversion tunnels, Figure 7. After the concrete dam has been constructed to a predetermined elevation, the right diversion tunnel will be permanently plugged at the vertical bend, Figure 3, and the riverflow will be diverted through the left tunnel. This flow will be controlled by three 7- by 10-foot high-pressure slide gates installed near the vertical bend, Figure 58. Unsymmetrical operation of these gates caused the flow to swing from side to side of the tunnel, Figure 59. For certain combinations of head, discharge, and gates in use, the jet leaves the flip bucket at an angle and impinges on the canyon wall. The 30-foot-long extension of the deflector was found necessary to prevent the jet from striking the canyon wall.

The flow from the revised flip buckets was very good. A small part of the jet from the right bucket struck the canyon wall at flows less than 50,000 cfs, but the impingement was not severe since the direction of flow and alinement of the walls were nearly parallel. At the larger discharges, the deflector directed the jet away from the canyon wall. In the left bucket, the deflector directed the flow away from the canyon walls at all discharges. The portion of the jet that impinged on the deflector rose vertically along the wall, in effect forming an L-shaped jet. This jet shape caused a concentration of the flow in the river at the impact point but flow conditions were satisfactory except at the maximum discharge. At the maximum discharge, the jet was compact at the point of impact and set up an eddying action that caused erosion of the riverbed; however, since the eddies did not extend upstream this was not considered objectionable.

The wave action depressing the jets downstream from the buckets also occurred with these revised buckets. The action was similar to that previously described except that the waves were higher, reaching heights equivalent to 50 or 60 feet midway between the buckets and about 15 feet high (in the form of slow surges) in the powerplant afterbay.

Fifth revision (recommended). --At this stage of the model investigations, excavation at the damsite had shown that the rock foundation was not as extensive as originally indicated and the small ridge of rock behind the river wall of the buckets could not be relied upon to take the hydraulic loads transmitted by the walls. Possible bucket modifications included placing the buckets on firm rock by moving the buckets upstream into the tunnels or reducing the load

3/Air and Hydraulic Model Studies of the Left Diversion Tunnel Outlet Works for Glen Canyon Dam, Report Hyd-468.

on the walls either by reducing their height or reducing the overall size of the buckets.

Temporary modifications of the buckets were made and tested to determine which of the above changes was most effective. These exploratory tests showed that the buckets could be moved upstream the necessary 20 feet, but that no significant changes should be made in their radius of curvature, angle of flip or length.

The amount of deflection on the left wall at the lip of the left bucket was increased from 4 to 7 feet to provide better protection against the jet impinging on the canyon wall. As determined from the left Diversion Tunnel Outlet Works tests, the 30-foot extension beyond the end of the bucket was retained, making the total amount of deflection at the end of the wall 12 feet 3 inches.

The exploratory tests also indicated that the river wall of the bucket could be reduced in height. When the portion of the wall above the tunnel springline was removed from the model bucket, reducing the wall height by more than 20 feet, flow from the river overtopped the wall and interfered with the jet during spillway discharges greater than 75,000 cfs. When the wall height was raised 5 feet above the springline, flow from the river did not overtop the wall at any spillway discharge. Although the depth of water in the bucket was greater than the height of the wall, the flow velocity was sufficiently high that very little lateral expansion of the jet occurred.

The buckets were rebuilt to incorporate most of the desirable features determined during the temporary modifications and discussed above. Both buckets were identical except for length of canyon wall, Figures 60-63, and consisted of the following features. The invert radius was 108.95 feet with the PC's located at the tunnel portals; the length of each bucket, from the PC to the lip, was 67.50 feet; and the lift or change in elevation was 23.43 feet. The outside walls converged 7 feet toward the centerline in 40 feet and the convergence started 30 feet downstream from the portals. The inside walls diverged 4 feet in a distance of 40 feet; the divergence was accomplished by an arc segment of a 202-foot radius circle starting 30 feet downstream from the tunnel portal. The outside wall of the left tunnel extended downstream 32.5 feet beyond the lip of the bucket at the same rate of convergence. The outside wall of the right tunnel and the inside walls of both tunnels terminated 2.5 feet downstream from the bucket lip. The tops of the inside walls of both tunnels were about 6 feet above the springline of the tunnels. The tops of the outside walls were 2 feet high at the portal and sloped upward on a 33.47 percent slope.

A total of 32 piezometers were installed in the left bucket--6 in the invert, 17 in the left wall, and 9 in the right wall, Figure 64.

The performance of these flip buckets was excellent in every respect. At maximum discharge, the jets leaving the buckets were very compact, and at their impact point the jets from the two buckets covered the width of the river channel in such a manner that there was no return flow along either bank, under the jets, or in the center of the channel. Some return flow occurred along the banks with the smaller discharges, but the eddies did not extend far enough upstream to erode the river banks or channel bottom in the tailrace area. Figures 65, 66, and 67 show flow conditions with the recommended buckets.

Top profiles of the jets for maximum discharge were obtained for the purpose of determining whether the powerlines in their proposed location over the river channel would be endangered by splash and spray. The profiles, shown on Figure 68, indicated that relocation of the powerlines was unnecessary.

Pressure measurements were obtained at the maximum discharge and three smaller discharges as an aid in the structural design of the buckets and to determine whether any cavitation pressures were present. These measurements indicated that the highest pressures would occur along the invert at Piezometer 1 at maximum discharge and would be equivalent to about 211 feet of water. The lowest observed pressure occurred at Piezometer 26, and was equivalent to 7.6 feet of water below atmospheric. The pressure readings are tabulated in Figure 64.

The performance of the fifth revised flip buckets was satisfactory in all respects and they were recommended for prototype installation.

River Outlets and Powerplant Afterbay

The river outlets and the powerplant afterbay tests are necessarily grouped together since flow from the river outlets affects the flow conditions in the afterbay. These investigations were concerned with dispersing the flow from the outlets with minimum flow disturbances in the afterbay and minimum riverbed erosion. The minimum size riprap protection in the afterbay area was also determined.

River outlets. --The river outlets are four 96-inch hollow-jet valves located on the left side of the river 150 feet downstream from the machine shop, Figures 2 and 69. The outlets will be used principally to maintain the minimum downstream riverflow before the powerplant is in operation and to control storage in the reservoir during the flood seasons after the right diversion tunnel is closed. In the latter instance, the valves will be used in conjunction with the tunnel plug outlet works. The valves will also be used to supply sufficient releases during floods which approach the magnitude of the ultimate design flood. The maximum discharge capacity of the four valves is only 15,000 cfs due to the velocity limitations in the conduit. The

comparatively large valves are needed because it might be neces-
sary to release the maximum discharge at very low heads during
initial operation. At high reservoir elevations the valves will be
operated only at partial openings. In the preliminary layout, the
valves were horizontal and parallel in plan with all centerlines at
the same elevation.

The jets from the valves, in effect, landed as a unit and caused
considerable disturbance at the point of impact, Figure 70. The
churning action of the jets eroded the riverbed at the point of impact
and displaced large amounts of material. The eroded material
moved in the direction of flow and formed a sandbar, semicircular
in plan, downstream from the jet impact area. After 1 hour of
operation (model time) this semicircular sandbar had extended
across the width of the river channel and had moved 200 to 300
feet downstream from the jet impact area. The height of the
deposited material was about 8 feet higher than the original bed.
As the sandbar built up, it turned part of the flow, causing eddies
to form on each side of the impact area. On the right side, a
clockwise eddy formed and moved upstream toward the tailrace,
passed in front of the powerhouse, then moved downstream, and
re-entered the area where the jets were striking. On the left side,
a counterclockwise eddy formed and moved toward the left canyon
wall, turned, flowed upstream along the wall, and re-entered the
jet impact area.

The riprapped apron in front of the powerhouse was represented
in the model by a concrete surface. Initially, the eddies carried
some of the eroded riverbed material onto this concreted surface;
as the action progressed, the eddy removed material from in front
of the concreted surface. The erosion in this area after about
3 hours operation of the outlets is shown on Figure 70. The over-
all severity of the erosion and the formation of large eddies indic-
ated that modifications of the flow pattern from the river outlets
were necessary.

To determine if the erosion pocket in the afterbay would eventually
stabilize, the deeply eroded areas adjacent to the concrete apron
were filled with sand and the downstream sandbar removed until
the riverbed was at elevation $3130^{\pm}$. The deep hole that was
eroded by the impact of the jets was not filled. The water level
in the model tailbox was slowly raised until the tailwater eleva-
tion was at 3144; then the river outlets were opened to discharge
15,000 cfs. Almost immediately the same eddy action started
and after a few minutes the flow pattern and eroded areas were
identical to those observed in the first test.

The concrete apron downstream from the powerhouse was removed
from the model and replaced with 3/4- to 3/8-inch gravel, repre-
senting 30- to 36-inch prototype riprap. Sand representing the
erodible material in the river was extended upstream to the riprap.

28

Operation at 15,000 cfs showed the same eddy patterns as observed in the previous tests. The erosion after 3 hours' operation also was similar; all of the loose bed material along the downstream edge of the riprap was removed by the eddies but none of the riprap was displaced.

To disperse the jets from the valves over a wider area, the valve alinement was modified by turning the three right-hand valves to the right. The left (No. 4) valve alinement was unchanged; No. 3 valve was turned 5° to the right, No. 2 valve 10°, and No. 1 valve 15°, Figure 71A. Spreading the jets helped the flow pattern considerably. After 5 hours' (model time) operation, the eddies and erosion were reduced over that observed with the original alinement after 1 hour's operation. There was no movement of the riprap in the afterbay area. However, the riverbed material that eroded from the impact area moved downstream and formed a sandbar across the river approximately on a line between the two flip buckets. When the river outlets were shut down and the only flow was through the powerplant, this bar became the water-level control and raised the water surface elevation at the powerhouse about 5.3 feet above normal tailwater elevation.

To further disperse the flow from the river outlets, the angle between the valves was increased an additional amount. The left valve was unchanged; the second, third, and fourth valves from the left were turned to the right of their original alinement 7-1/2°, 15°, and 22-1/2°, respectively, Figure 71B.

The model was then operated with 15,000 cfs through the valves, 32,000 cfs through left spillway, and 24,000 cfs through powerhouse; the jets were well dispersed, Figure 71. Eventually the flow pattern became the same as described in the original tests, but since the flow was more dispersed the length of time required to attain this flow pattern was longer. The riverbed erosion at the end of 10 hours' model operation was similar to that obtained in the previous test; the eroded material formed a sandbar far downstream from the flip buckets, Figure 72. The top of the sandbar was at elevation 3047 and caused the tailwater elevation to be 5 feet above normal during subsequent runs with only the powerhouse in operation. Some of the riverbed material that was disturbed by the jets moved upstream with the clockwise eddy and was deposited on the riprap apron of the afterbay, Figure 72. Examination showed that this material came from the area along the right riverbank between the riprap and the right tunnel portal. It was estimated that close to 35,000 cubic yards of material moved into the tailrace.

The deposition of riverbed material on the riprap apron might entail costly maintenance problems; therefore, methods of preventing the deposition of material were investigated in the model. The first method consisted of preexcavating the area along the right riverbank,

29

where previous model tests had indicated that most of the river material had originated. Figures 73 and 74A show the outline of the preexcavated area.

The model was operated (left spillway--32,000 cfs, outlet works--15,000 cfs, and powerplant--24,000 cfs) for 8 hours and examined to determine the amount of riverbed material that had moved into the tailrace area. Very little material was deposited so the operation was continued for an additional 8 hours. Figure 74B shows the appearance of the tailrace after 16 hours' operation. No additional riverbed material had moved into the tailrace. This corrective method involved the removal of approximately 30,000 cubic yards of riverbed material a costly undertaking.

The second method of preventing the deposition of material on the riprap was by regulating the discharge through the valves. This method consisted of operating the two left-hand valves (No. 4 and No. 3) for as long as possible to direct the outlet flow downstream and to limit the operation of the two right-hand valves to only when large releases were necessary. For the model investigation of this method, the riverbed was reformed as shown in Figure 74A and the model was operated with 24,000 cfs through the powerplant, 32,000 cfs through the spillway and 7,500 cfs through the two left-hand valves. At the end of 7 hours' operation, an equivalent of approximately 5,000 yards of material had moved onto the riprap; most of the movement had taken place during the first 2 hours of operation so a longer test was deemed unnecessary. Figure 74C shows the tailrace area after this test.

This method of preventing excessive sedimentation in the tailrace was considered satisfactory. Although it was less effective than the first method, the second method was adopted over the first method which was considered too costly for the improbable flow conditions that would require operation of all four outlets.

The riprap in the tailrace had been subjected to over 30 hours' operation during these tests without being disturbed. This indicated that the size of riprap in this area might be reduced. Originally the specifications called for 30- to 48-inch-diameter rock; in the model these were represented by stones 3/8- to 3/4-inch in diameter. This rip-, rap was replaced by stones 1/4- to 3/8-inch in diameter, representing prototype rock 18 to 24 inches in diameter. The smaller size riprap was in place during the two tests to determine a method of reducing the sedimentation in the tailrace. There was no movement of the riprap during these tests; some settlement or consolidation was apparent but easily identified individual stones were noticed in the same places after each test.

These tests showed that the riprap in the tailrace could consist of rock 24 to 30 inches in diameter, rather than the 30- to 48-inch-diameter rock originally specified. However, because of subsequent design and cost considerations, a concrete slab was placed in the powerplant tailrace instead of the riprap.

Water surface drawdown--Spillway operation.--A major concern relative to the operation of the structure was the lowered tailwater elevation at the powerhouse during operation of the spillways. The reduction in water surface elevation or "drawdown" was caused by the ejector action of the jets striking the river and forcing the water downstream; the upstream water was drawn into the jet impact area to replenish the ejected water, resulting in a depressed upstream water level.

Approximately 3,000 feet of river channel downstream from the powerhouse was represented in the model. The estimated solid rock boundary of the river channel had been placed in concrete and the sand and gravel of the riverbed were represented by sand placed on the concrete. In this manner, the stable riverbed and the erodible bedload deposits were represented. The estimated solid rock outline and the extent of sand deposits in the river were obtained from field drawings. The river outline was established from specification drawings.

In preparing the model for determining the water surface drawdown, the sand bed was leveled and lightly compacted, and the riprap cover was placed in the tailrace. The operating procedure was to discharge 24,000 cfs through the powerhouse and to pass a known flow through the spillways. The tailwater elevation corresponding to the combined flows was set by the tailwater control gate at the downstream end of the tailbox. When the water levels had stabilized the water surface elevation in the tailrace 20 feet (prototype) downstream from the powerhouse was recorded. The difference between the recorded and the normal tailwater elevations was the amount of drawdown.

The water surface did not have a uniform slope between the two stations; the surface sloped slightly downward from the powerhouse to the point of impact of the jets where there was an abrupt increase in the water level and an area of extreme turbulence followed by a mild slope from the turbulent area to the tailgate control.

The water surface drawdown was determined for the range of spillway flows from no flow increasing to the maximum discharge in increments of 50,000 cfs, and then in decreasing increments to no spillway flow. The fluctuation in water surface at the powerplant was also measured. The drawdown in the increasing-flow cycle was greater than that recorded in the decreasing-flow cycle because of the difference in riverbed erosion.

Three theoretical tailwater elevation curves were available, Figure 75. These curves were contained in the report, "Tailwater and Degradation Studies--Colorado River Below Glen Canyon Dam,"

prepared by the Hydrology Branch. The three curves were (1) initial conditions, (2) after channel degradation, (without Marble Canyon), and (3) with Marble Canyon Dam, (no channel degradation). The first curve represented initial operation without backwater effects from Marble Canyon and before channel degradation downstream from Glen Canyon Dam. The second curve assumed that clear water releases from Glen Canyon had caused downstream channel degradation, resulting in a lower water surface elevation. The third curve assumed that backwater from Marble Canyon Dam affected the tailwater elevation and no downstream channel degradation had occurred. The third curve was used in the tests to determine the water surface drawdown.

The tailwater elevations at the two stations, the water surface drawdown between the two stations, and the water surface fluctuation at the powerplant are shown on Figure 76.

Before erosion the maximum drawdown occurred for a combined flow of 300,000 cfs; the amount of drawdown was 30 feet. The maximum water surface fluctuation of 5 feet occurred during the maximum discharge.

A water surface drawdown curve was also obtained without the sand placed on the concreted solid rock outline of the riverbed. This curve was almost identical with the drawdown curve after degradation. One run was made at maximum discharge with the downstream tailwater elevation corresponding to Curve 2 on Figure 75 (after channel degradation, without Marble Canyon). The controlled downstream and observed upstream tailwater elevations were 5 feet lower than for the previous tests, suggesting that the amount of water surface drawdown would be the same for any of the three tailwater conditions.

One measurement was made with the left spillway discharging 138,000 cfs and no flow through the right spillway. For the three tailwater conditions, the water surface drawdown was slightly greater than when the combined discharge of both spillways was 138,000 cfs.

The same series of water surface drawdown tests were made with 15,000 cfs discharging through the river outlets, 24,000 cfs through the powerhouse, and both spillways operating. In general, the operation of the river outlets increased the drawdown about 1 foot; this was true for the full range of spillway discharges and for before and after riverbed erosion.

Limited investigations were made to determine the effect on the upstream water surface elevation after the spillways had operated at a combined flow of less than 100,000 cfs for relatively short time periods. The model tests consisted of preparing the riverbed to represent the predegradation conformation (8-10 inches of sand on top of the concrete). Flow included the spillway discharge plus 24,000 cfs through the powerhouse and, in some tests, 15,000 cfs through the river outlets. For spillway discharges up to 75,000 cfs, the spillway jets eroded the riverbed material forming a sandbar downstream from the impact area. The location and size of the sandbar depended on the spillway discharges. After the spillway flow was shut down, the sandbar controlled the upstream water level for powerhouse and outlet flows and for all spillway discharges less than the discharge that had formed the sandbar. This sandbar increased the water surface elevation at the powerhouse by 5 feet for lesser spillway flows and for powerhouse operation only.

A sandbar also formed for spillway discharges greater than 75,000 cfs. It was not possible to record conclusive tailwater data in the model because the sandbar moved rapidly downstream. After 2 to 3 hours of model operation, the sandbar had moved beyond the model tailgate and no longer controlled the tailwater level. These results are only qualitative because the sandbar would control the tailwater for discharges above 75,000 cfs if a longer reach of the downstream river channel had been included in the model.

Water surface drawdown in powerplant afterbay. --A cost analysis indicated that it would be less expensive to place an 8-inch-thick concrete apron in the powerhouse afterbay rather than the riprap. To assist the designer in determining the number of weep holes necessary to relieve uplift pressures on the slab, water surface profiles for various operating conditions were measured.

Five electronic water-level measuring gages were placed on the center of the river channel between the powerhouse and the downstream tailwater control. Water surface variations at the five stations were simultaneously recorded on a Sanborn recorder so that the change in water surface with respect to time could be obtained. The water-level gages were arranged so that gages one through five were 53, 96, 210, 340, and 3,000 feet downstream from the powerhouse. The concrete apron will extend about 250 feet downstream from the powerhouse.

Three different operating conditions were tested. In Test 1, the powerplant was operating at 24,000 cfs with the tailwater stabilized at elevation 3146. The four river outlets were opened over a period

33

of 8 minutes (all values are prototype), ultimately discharging 15,000 cfs, giving a total flow of 39,000 cfs. Operation was continued at this flow for about 7 hours. The recorded water surface profiles with respect to time, Figure 77, showed that at Stations 1 and 2 the water surface dropped about 1 foot in the first 45 minutes, dropped another foot in the next 75 minutes, and stable for the remainder of the test. The water surface at Station 3 dropped 1 foot before the valves were fully opened, increased to the original elevation during the ensuing 15 minutes, then gradually dropped about 5 feet during the next 200 minutes, and then fluctuated between 4 and 5 feet for the remainder of the test. The water surface at Station 4 dropped 3 feet as the valves were opening, recovered about 1-1/2 feet during the next 10 minutes, then gradually dropped about 3-1/2 feet over the ensuing 60 minutes, at which time the measuring device became inoperative due to a sandbar that formed directly under it. The water surface at Station 5 raised 1 foot during the time the valves were being opened, then remained constant for the duration of the test, which represented the normal rise in tailwater elevation for this increase in discharge. This test showed that for this operating condition, the most critical period for uplift would be while the valves were being opened and the drop in the water surface at the end of the apron would be about 2 feet.

In Test 2 the powerhouse was discharging 24,000 cfs with the tailwater stabilized at elevation 3146. The four spillway gates were opened at the rate of 2 feet per minute (fpm) until the combined spillway flow was 15,000 cfs. This flow was used because it was the minimum discharge at which the jets swept out of the flip buckets. This operation was continued for about 2 hours; then the spillway flow was increased to 30,000 cfs and continued for about 1-1/2 hours. The spillways were then slowly closed over a period of about 1 hour; the test was continued for an additional hour with only the powerhouse operating.

The water surface profiles, Figure 77, indicated that the water level at Station 1 rose less than 1 foot in the first hour and then remained constant until the spillway flow ceased. At Stations 2, 3, and 4 the water level rose 1-1/2 feet in the first hour, remained constant until the spillway flow increased to 30,000 cfs, then dropped about 0.25 foot and remained constant until the spillway gates were closed. The water surface level at Station 5 increased 1.5 feet as the spillway discharge increased to 15,000 cfs, remained constant until the spillway flow was increased to 30,000 cfs when the water level raised an additional 1.5 feet. The water level at Station 5 dropped as the spillway flow was shut off.

This test indicated that there should be no uplift problem during this operating condition.

34

In Test 3, the powerhouse was discharging 24,000 cfs with the tail-water stabilized at elevation 3146. The left spillway gates were slowly opened until the spillway discharge was 29,500 cfs, representing the maximum discharge of the tunnel plug outlet works. Operation was continued at this flow until the tailwater had stabilized; then, the powerhouse flow was slowly shut down. The tailwater was again allowed to stabilize; then the river outlets were slowly opened until they were discharging 15,000 cfs. This operation was continued for approximately 80 minutes; then the river outlets were slowly closed.

The recorded water surface profiles, Figure 77, showed that the water level at Stations 1 through 4 would increase about 1 foot as the spillway flow increased. When the spillway flow reached 29,500 cfs, the water level slowly dropped about 1.5 feet over a period of 45 minutes. When the powerhouse flow was shut off, the water level at Stations 1 through 4 dropped about 4 feet in a period of 10 minutes. The water level was stable at these stations until the river outlets were opened. As the river outlets were opened, the water level dropped 2 feet at Stations 1 and 2 and 5 feet at Stations 3 and 4. During the first 25 minutes of the river outlet operation, the water level rose about 1.5 feet at Stations 1 and 2 and 2.5 feet at Stations 3 and 4. When the river outlets were closed, the water level rose about 2.5 to 3.5 feet. The water level at Station 5 increased 3 feet as the spillway flow increased, dropped 2 feet when the powerplant was closed, and rose 1.5 feet when the river outlets were opened.

Test 3 indicated that for this operating condition the slab should be designed to provide for a water pressure differential of 3 feet when the river outlets were initially opened.

TABLE I
PROTOTYPE SPILLWAY VELOCITY COMPUTATIONS

Sta. 20 + 00

E.G.L. + Losses (El. 3712.0)

E.G.L.

Head loss "h_f"

Sta.

Sta.

Velocity head "h_v"

d_n

El.

$d_n \cos \theta$

Tunnel Portal

Sta. 26 + 11.72

$\theta = 55°$

El. 3137.4

DATA

1. Max. W.S. El. 3712.0.
2. Max. Q = 142,000 c.f.s.
3. Critical depth at Sta. 20 + 32.33.
4. Roughness coefficient "n" estimated to be 0.014.
5. For diameter of tunnel see figure.

FORMULAS

$$V = \frac{Q}{A} \; ; \quad h_v = \frac{v^2}{2g} \; ; \quad S = \frac{n^2 v^2}{2.208 \, r^{4/3}} \; ; \quad h_f = S_{av} L$$
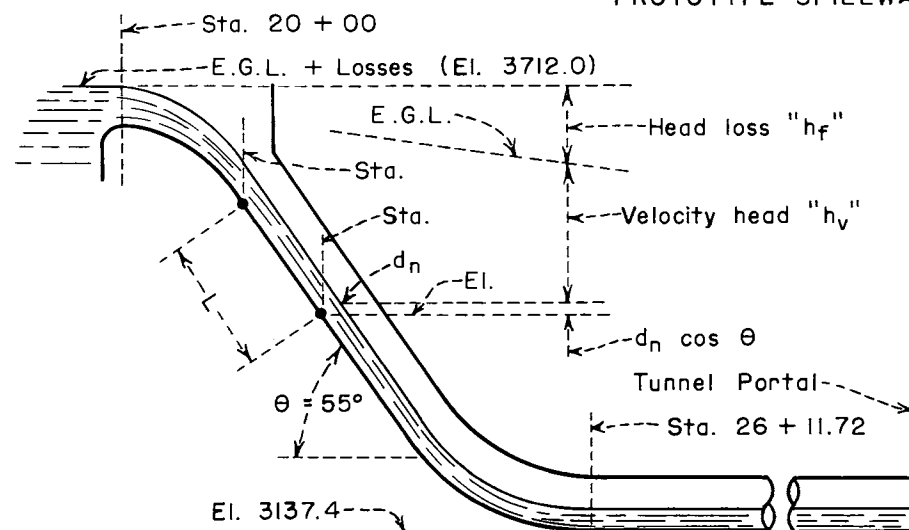
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Station	Elevation	d_n	$d_n \cos \theta$	A	V	h_v	r	S	S_{av}	L	h_f	Σh_f	$d_n \cos \theta$ $+h_{v_1}+ \Sigma h_f$	Elevation + ⑭
20 + 32.33	3642.9	46.08	46.08	1843.2	38.52	23.04	13.95	0.003924	—	—	—	—	69.12	3712.0
21 + 46.38	3543.5	34.91	20.02	1462.5	97.09	146.37	14.80	.025939	0.014931	153.89	2.30	2.30	168.69	3712.19
22 + 10.91	3451.3	30.20	17.32	1148.7	123.62	237.30	13.23	.043340	.034640	112.49	3.90	6.20	260.82	3712.12
22 + 75.43	3359.2	28.45	16.32	982.7	144.50	324.23	12.16	.066319	.054830	112.49	6.17	12.37	352.92	3712.12
23 + 26.24	3286.6	26.30	15.09	894.3	158.78	391.41	11.73	.083893	.075106	88.58	6.65	19.02	425.52	3712.12
24 + 50.20	3177.1	24.17	21.42	810.2	175.26	478.92	11.28	.108154	.096025	167.38	16.07	35.09	535.43	3712.63
26 + 11.72	3137.4	23.80	23.80	793.4	178.97	497.36	11.19	.113680	.110917	167.38	18.56	53.65	574.81	3712.21
28 + 11.72	3136.7	24.25	24.25	811.9	174.89	474.95	11.28	.107257	.110468	200.00	22.09	75.74	574.94	3711.64
32 + 11.72	3135.3	25.15	25.15	848.9	167.27	434.46	11.48	.095886	.101572	400.00	40.63	116.37	575.98	3711.28
34 + 51.72	3134.2	25.70	25.70	870.8	163.08	412.97	11.60	.089838	.092862	240.00	22.29	138.66	577.33	3711.53

Column 3 assumed

TABLE 2
I : 63.48 MODEL SPILLWAY VELOCITY COMPUTATIONS

--- Sta. 0.000
--- E.G.L. + Losses (El. 100.000)
E.G.L.
--- Head loss "h_f"
--- Sta.
--- Sta.
--- Velocity head "h_v"
--- dn.
--- d_n cos. Θ
L
$\Theta = 55°$
Tunnel Portal
--- Sta. 9.633
El. 90.950

DATA

1. Max. W.S. El. assumed to be 100.000.
2. Max. Q = 4.422 c.f.s.
3. Critical depth at Sta. 0.510'.
4. Roughness coefficient "n" estimated 0.010.
5. For diameter of tunnel use model scale on fig.

FORMULAS

$$V = \frac{Q}{A} \; ; \; h_v = \frac{V^2}{2g} \; ; \; S = \frac{n^2 V^2}{2.208 \, r^{4/3}} \; ; \; h_f = S_{av} L$$
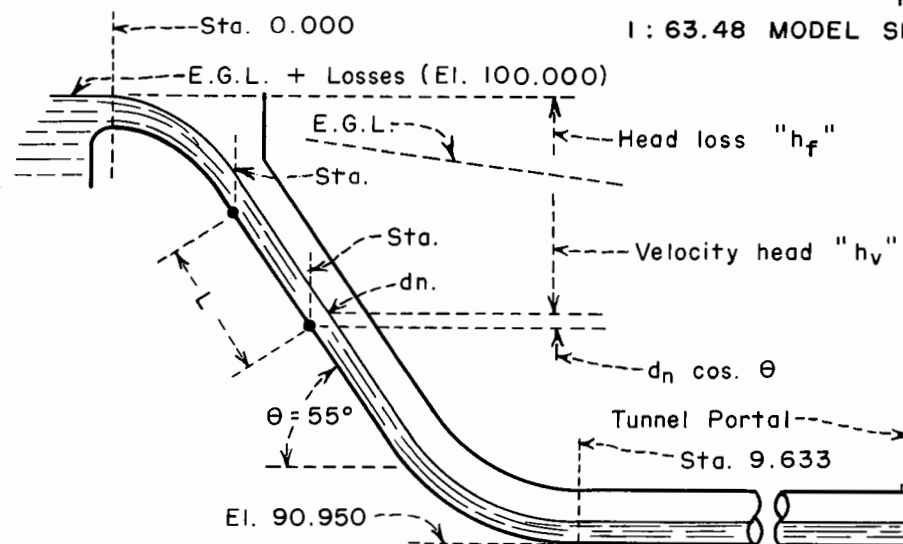
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Station	Elevation	d_n	$d_n \cos \Theta$	A	V	h_v	r	S	S_{av}	L	h_f	Σh_f	$d_n \cos \Theta$ + h_v + Σh_f	Elevation + ⑭
0.510	98.911	0.726	0.726	0.457	4.84	0.363	0.220	0.0080	—	—	—	—	1.089	100.00
2.305	97.346	.553	.317	.365	12.12	2.28	.234	.0461	.0270	2.423	0.0655	0.0655	2.662	100.01
3.321	95.895	.480	.257	.288	15.35	3.66	.209	.0861	.0661	1.771	.1170	.1825	4.099	99.99
4.337	94.444	.455	.261	.248	17.83	4.94	.192	.1300	.1080	1.771	.1913	.3738	5.575	100.02
5.138	93.301	.423	.243	.227	19.48	5.89	.187	.1607	.1453	1.395	.203	.5768	6.710	100.01
7.092	91.577	.392	.347	.208	21.26	7.02	.180	.2014	.1810	2.636	.477	1.0538	8.421	100.00
9.633	90.950	.390	.390	.207	21.36	7.09	.180	.2033	.2023	2.636	.534	1.5880	9.068	100.02
11.995	90.941	.402	.402	.215	20.57	6.57	.182	.1857	.1945	2.362	.459	2.0468	9.019	99.96
12.784	90.939	.408	.408	.218	20.28	6.39	.183	.1793	.1825	0.789	.144	2.1908	8.989	99.93
19.082	90.917	.438	.438	.237	18.66	5.41	.189	.1453	.1623	6.299	1.022	3.2128	9.061	99.98
22.862	90.900	.460	.460	.250	17.69	4.86	.193	.1270	.1361	3.780	.514	3.7268	9.047	99.95

Column 3 assumed

FIGURE I
REPORT HYD. 469

BEAVER · PIUTE · WAYNE · Green River

Marysvale · Milford · Adamsville · Beaver · Minersville · Circleville · Loa · Bicknell · Teasdale · CAPITOL REEF NAT'L. MON. · Antimony

IRON · GARFIELD · SAN

Lund · Parowan · Panguitch · Boulder · Escalante · Fremont River

Newcastle · Cedar City · Hatch · BRYCE CANYON NAT'L. PARK · Escalante River

WASHINGTON · Pintura · Anderson Jct. · La Verkin · Orderville · Glendale · Mt. Carmel · ZION NAT'L. MON. · ZION NAT'L. PARK · KANE · WAHWEAP DEPOSIT · JUAN

St. George · Hurricane · Kanab · Wahweap Cr. · RAINBOW BRIDGE NAT'L. MON. · San Juan River · Mexican Hat

UTAH · ARIZONA

Lees Ferry · Rainbow Lodge · GLEN CANYON DAM · Page · Marble Canyon · INDIAN

Jacob Lake · Bitter Springs · Copper Mine Trading Post · Shonto Spring · Kayenta · RESERVATION

MOHAVE · COLORADO · GRAND CANYON NAT'L. PARK · Gap · Tuba City · Tonalea · HOPI · INDIAN · Pinon

GRAND CANYON NAT'L. PARK · Grand Canyon · Little Colorado River · Cameron · Oraibi · RESERVATION

Peach Springs · Cameron · Mt. Eldon · NAVAJO · Greasewood Springs

MOHAVE · COCONINO · Seligman · Sunrise · Dilkon · INDIAN RESERVATION

Ashfork · Williams · Flagstaff · Winona · Winslow · Colorado River

SCALE OF MILES
10 0 10 20 30

KEY MAP

NEVADA · UTAH · Salt Lake City · Wendover · Provo · Reno · Delta · Greenriver · Moab
CARSON CITY · SACRAMENTO · Stockton · SAN FRANCISCO · Merced · Fresno · Milford · Marysvale
Caliente · Cedar City · Kanab · Lees Ferry · Marble Canyon · Page · GLEN CANYON DAM
Las Vegas · Grand Canyon · Williams · Flagstaff · Winslow
Bakersfield · Mohave · Barstow · Kingman · LOS ANGELES · Needles · Wickenburg · PHOENIX
CALIFORNIA · ARIZ.

	4-18-58	ADDED LOCATIONS OF TESTED AGGREGATE DEPOSITS
	4-1-57	ADDED THE LOCATION OF THE COMMUNITY SITE AT GLEN CANYON DAM
	12-14-56	ADDED OUTLINE OF RESERVOIR

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.— GLEN CANYON UNIT - ARIZ.- UTAH
GLEN CANYON DAM
LOCATION MAP

DRAWN B.F.W. SUBMITTED
TRACED B.F.W. RECOMMENDED
CHECKED APPROVED

DENVER, COLORADO — AUG. 1, 1956

557-D-29

FIGURE 2
REPORT HYD. 469

Intersection of ℄ spillway with axes of crest-Sta. 20+00
N 2,161,414, E 627,178

2-40'x52.5' Radial gates

℄ Service road

Diversion tunnel plug

El. 3715

E 627,500

℄ 41' Dia Diversion tunnel

Spillway tunnel - S 7°00' E

3800

N 2,160,500

E 2,160,000

Deflector bucket - El. 3130
Portal, Sta. 36+96

Gate hoist bridge

Guardrail

Spillway approach channel, El. 3635

Service area El. 3730

Chain-link fence and gate

Gate erection platform

℄ Service road to power plant

Machine shop

Service bay

Portal area El. 3188.5

River outlets valve structure

COLORADO RIVER

River outlets trashrack structures

Gate service shaft

Elevator tower

Vista canopy

Reference plane for dam - S.19°43'36.34" E.

900 R

Reference plane for power plant
S 21°00'00" E.

Power plant

Colorado River bridge

El. 3125

El. 3132

Penstock trashrack structures

Crane rails

Axis Sta. 9+80

10

Portal, Sta. 35+22
Deflector bucket

Axis Sta. 10+00
N 2,160,875
E 626,580

Axis of dam

15

Vista canopy

Elevator tower

El. 3126.00

SCALE OF FEET

Gate hoist and stems storage platform

Block number

Service bridge

℄ Spillway Tunnel - S 31°00' E

Central cable tunnel

Parking Area

El. 3860±

El. 3540±

℄ 41' Dia. Diversion tunnel
Chain-link fence and gate
Diversion tunnel plug

El. 3750

El. 3818±

Switchyard El. 3885±

Guardrail

Service area El. 3730

Spillway approach channel, El. 3635

Intersection of ℄ spillway with axes of crest-Sta. 20+00
N 2,161,096, E 625,748

2-40'x52.5' Radial gates

Gate hoist bridge

℄ Service road

El. 3828±

U.S. highway 89

Parking area

℄ Switchyard service road

	REVISED TO CONFORM WITH CONSTRUCTION DRAWINGS
1-22-62	REVISED AND REDRAWN
3-8-57	REVISED ALINEMENT OF SPILLWAY APPROACH CHANNELS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.- GLEN CANYON UNIT - ARIZ.- UTAH

GLEN CANYON DAM AND POWER PLANT
PLAN

DRAWN ... SUBMITTED ... O.L. Rice
TRACED ... RECOMMENDED ... K.B. Keener
CHECKED ... APPROVED ... Grant Bloodgood

DENVER, COLORADO, NOVEMBER 19, 1956

557-D-72

PLAN

ELEVATION

SECTION A-A

SCALE OF FEET

GLEN CANYON DAM SPILLWAYS
1.88 SCALE MODEL
DIVERSION STUDY LAYOUT

FIGURE 5
REPORT HYD. 469

Figure 6
Report Hyd-469

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model

FIGURE 3
REPORT HYD. 469

UPSTREAM ELEVATION
DEVELOPED ALONG AXIS

SECTION THRU RIVER OUTLETS

SECTION THRU PENSTOCK AND POWER PLANT

AREA, CAPACITY, AND DISCHARGE CURVES

RESERVOIR CAPACITY-1,000,000 A.F.
RESERVOIR AREA-10,000 ACRES
SPILLWAY DISCHARGE-10,000 C.F.S.
RIVER OUTLETS DISCHARGE-1,000 C.F.S.

SECTION THRU RIGHT SPILLWAY TUNNEL

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON DAM AND POWER PLANT
ELEVATION AND SECTIONS

DRAWN... SUBMITTED... O.L. Rice
TRACED... RECOMMENDED... K.B. Keener
CHECKED... APPROVED... Grant Bloodgood

DENVER, COLORADO, NOV. 19, 1958   557-D-73

FIGURE 4
REPORT HYD. 469

¢ Service bridge
¢ Operating bridge
El. 3730.00
Intersection of axes of crest -
Sta. 20+00 (Along ¢ of tunnel)
Max. W.S. El. 3711
Nor. W.S. El. 3700
Spillway intake
40'x 52.5' Radial gate
Crest - El. 3648.00
El. 3638.122
Sta. 20+45.000
$x^2 = -205y$
P.T. - El. 3543.470
Sta. 21+46.385
El. 3506.677
48.250' Dia.
Sta. 21+72.148
55° 00'00"
El. 3349.331
41.000' Dia.
Sta. 22+82.323
P.C.- El. 3286.611
Sta. 23+26.240
350.000' R.
41.000' Dia.
Sta. 26+12.943
El. 3487.363

P.I.- El. 3138.000
Sta. 24+30.298
Sta. 26+11.718
¢ Tunnel
P.T. - El. 3137.365
Slope 0.0035
Downstream portal
El. 3133.570
El. 3134.179 (Right tunnel)
108 - 11 3/8" R.
El. 3157.00
El. 3157.61 (Right spillway)
El. 3130.00
Deflector bucket
Sta. 36+96.00
Sta. 35+22.00
(Right tunnel)

PROFILE - LEFT SPILLWAY
(RIGHT SPILLWAY SIMILAR)

REFERENCE DRAWINGS

SPILLWAYS:-
SPILLWAY APPROACH CHANNELS - EXCAVATION _557-D-442
SPILLWAY INTAKE STRUCTURES - EXCAVATION_ _557-D-510
RIGHT AND LEFT TUNNELS - ELEVATIONS AND SECTIONS_ _ _ _ _ _ _ _ _ _ _ _ _ _ _ _ _557-D-444
RIGHT AND LEFT TUNNELS - GROUTING AND DRAINAGE DETAILS_ _ _ _ _ _ _ _ _ _ _ _557-D-445
INTAKE STRUCTURES - SHEET 1 OF 2 _557-D-601
INTAKE STRUCTURES - SHEET 2 OF 2 _557-D-602
HOIST BRIDGES - PLAN AND SECTIONS_ _557-D-678
DISCHARGE CURVES_ _557-D-1386
40'X 52.5' RADIAL GATE - GENERAL INSTALLATION _557-D-777
RADIAL GATE HOIST - INSTALLATION_ _557-D-1478

RIGHT SPILLWAY:-
DOWNSTREAM PORTAL - DEFLECTOR BUCKET - PLAN AND SECTIONS - SHEET 1 OF 2_ _ _ _ _557-D-1982
DOWNSTREAM PORTAL - DEFLECTOR BUCKET - PLAN AND SECTIONS - SHEET 2 OF 2_ _ _ _ _557-D-1983
TUNNEL - INVERT REPAIRS_ _557-D-2807

LEFT SPILLWAY:-
DOWNSTREAM PORTAL - DEFLECTOR BUCKET - PLAN AND SECTIONS - SHEET 1 OF 2_ _ _ _ _557-D-2099
DOWNSTREAM PORTAL - DEFLECTOR BUCKET - PLAN AND SECTIONS - SHEET 2 OF 2_ _ _ _ _557-D-2100

NOTE
For sheet 1 of 2, see Dwg. 557-D-443.

1-31-64 DELETED REFERENCE TO DWG. 557-D-2806

ALWAYS THINK SAFETY

GLEN CANYON DAM SPILLWAYS

1:88 SCALE MODEL

PRELIMINARY TAILWATER CURVES

Figure 9
Report Hyd-469

Discharge = 30,000 cfs
T. W. Elev. = 3043.6

Discharge = 65,000 cfs
T. W. Elev. = 3048.2

Discharge = 65,000 cfs
T. W. Elev. = 3053.1

GLEN CANYON DAM SPILLWAYS

1:88 Scale Model
Diversion Studies
Flow Conditions with Right Tunnel Operating

Q = 60,000 cfs   T. W. Elev. = 3047.7

Q = 130,000 cfs   T. W. Elev. = 3054.5

Q = 60,000 cfs   T. W. Elev. = 3052.4

Q = 130,000 cfs   T. W. Elev. = 3064.0

GLEN CANYON DAM SPILLWAYS

1:88 Scale Model
Diversion Studies
Flow Conditions with Both Tunnels Operating

Figure 10
Report Hyd-469

FIGURE 11
REPORT HYD. 469

TYPICAL PLAN

TYPICAL SECTION A—A

SECTION B-B
25° · BUCKET

SECTION B-B
30° BUCKET

SECTION B-B
35° BUCKET

SECTION B-B
40° BUCKET

SECTION B-B
45° BUCKET
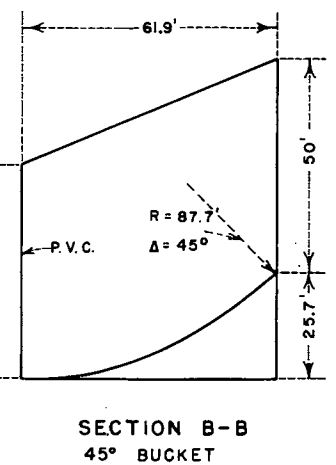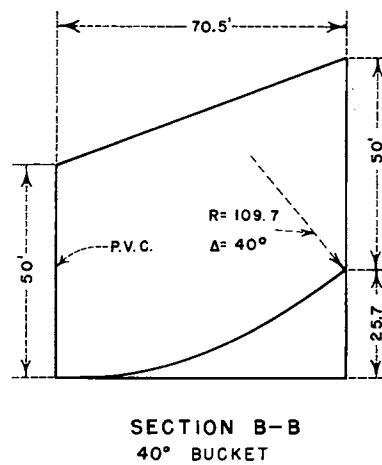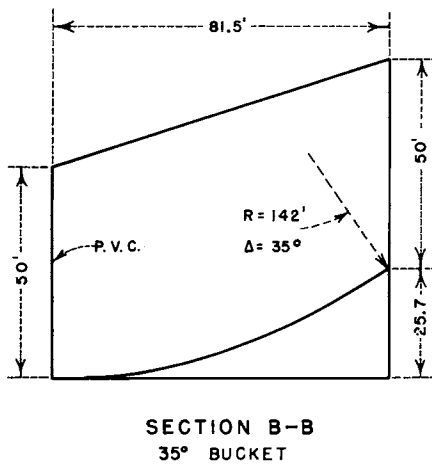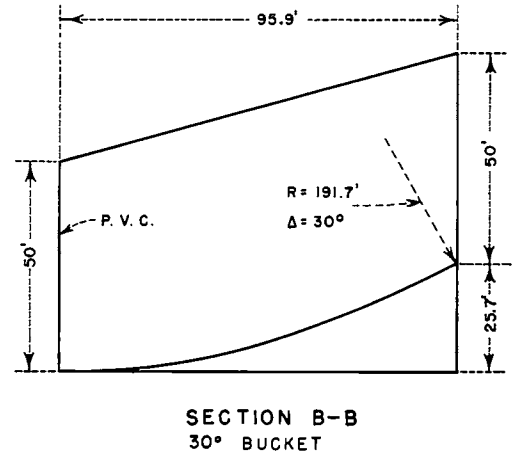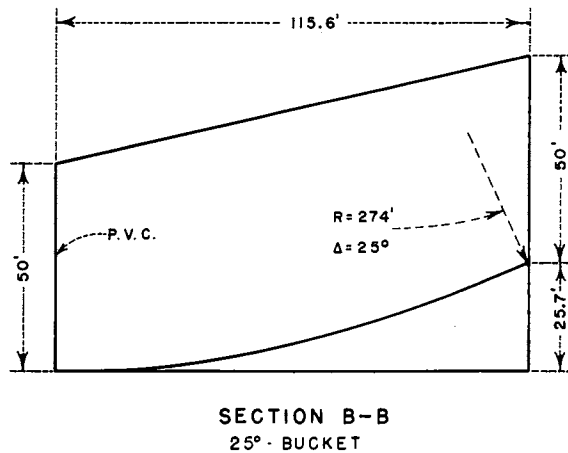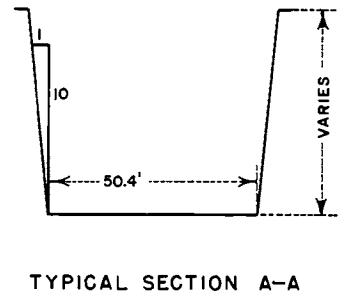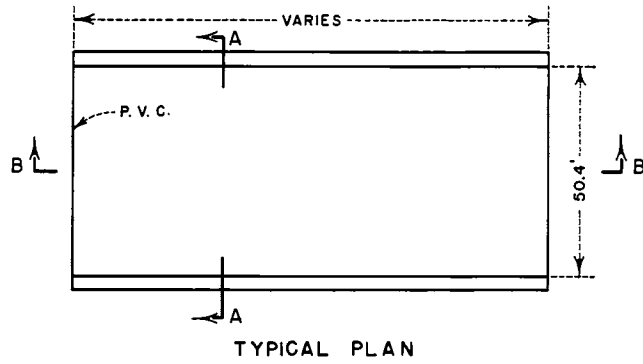
GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRELIMINARY FLIP BUCKETS

FIGURE 12
REPORT HYD. 469

NOTES

MARBLE CANYON T.W.   LEE'S FERRY T.W.
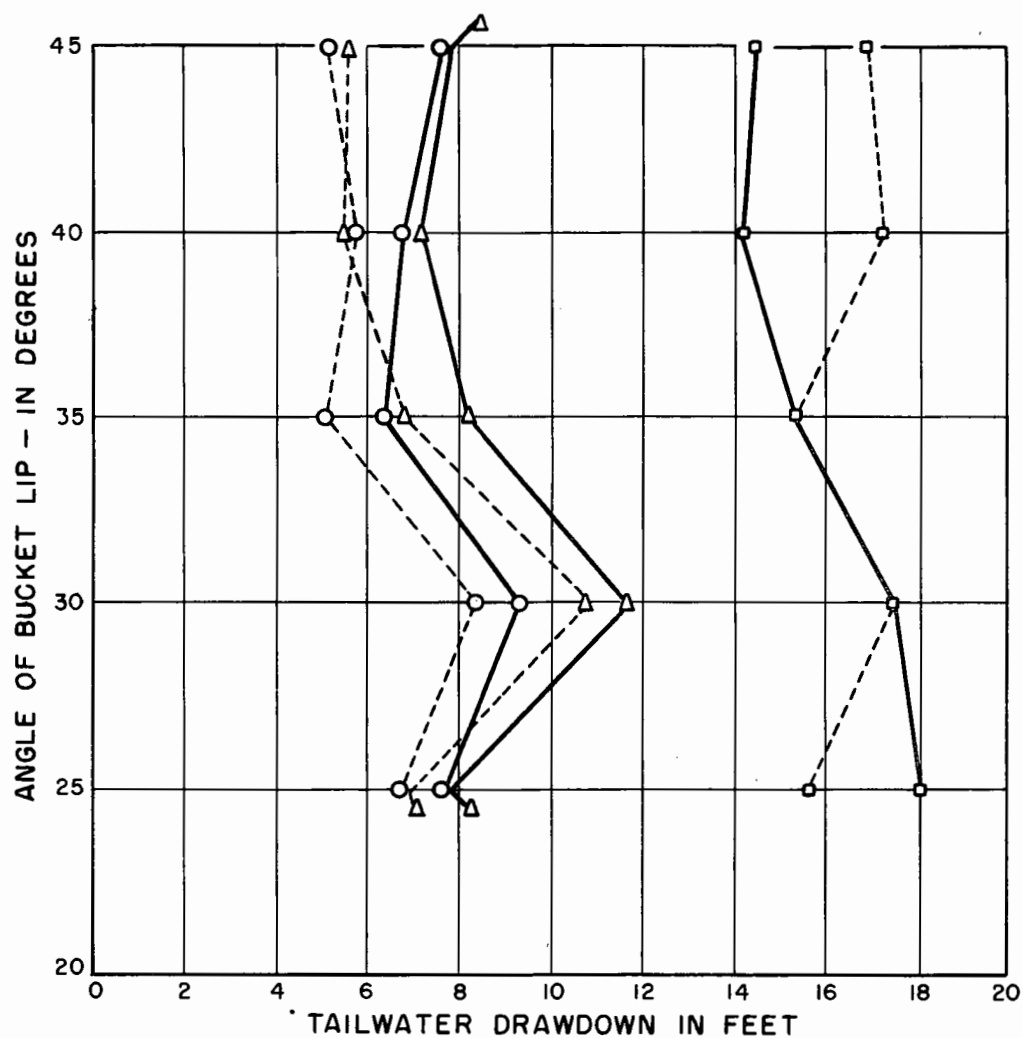
	MARBLE CANYON T.W.	LEE'S FERRY T.W.
RIGHT BUCKET	○——————○	○- - - - - - - -○
LEFT BUCKET	△——————△	△- - - - - - - -△
BOTH BUCKETS	□——————□	□- - - - - - - -□

Drawdown is the difference in T.W. elevation measured
at the powerhouse tailrace and 1800' ± down —
stream in the center of the channel.

GLEN CANYON DAM SPILLWAYS
1:88 SCALE MODEL
TAILWATER DRAWDOWN FOR VARIOUS LIP ANGLES

Figure 13
Report Hyd-469

25° Flip Angle

35° Flip Angle

45° Flip Angle

Discharge   = 121,000 cfs each **tunnel**
T. W. Elev. = 3182. 0

GLEN CANYON DAM SPILLWAYS

1:88 Scale Model
Preliminary Flip Bucket Operation

GLEN CANYON DAM

Project photographs of undercutting of **right**
canyon wall during river diversion Dec. 1959
Jan. 1960

GLEN CANYON DAM SPILLWAYS

Project photographs of canyon wall failure
Right Diversion Tunnel Portal.   June 13, 1960

FIGURE 7
REPORT HYD. 469

GLEN CANYON DAM DIVERSION TUNNELS
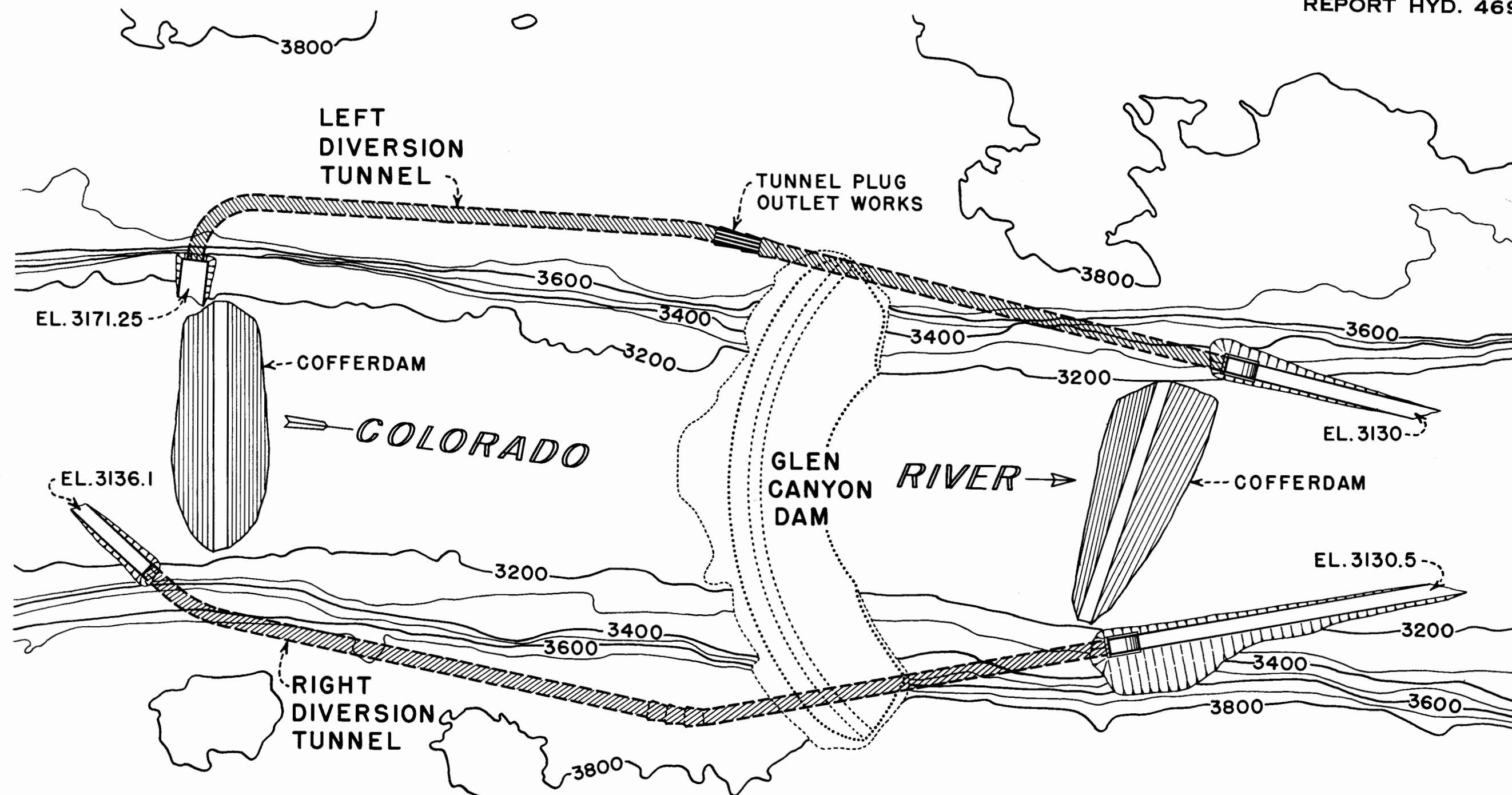
Figure 16
Report Hyd-469

15,000 cfs

50,000 cfs

A. Preliminary Side Channel

15,000 cfs

50,000 cfs

B. Modified Side Channel

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow at right diversion tunnel with side channels.

FIGURE 17
REPORT HYD. 469
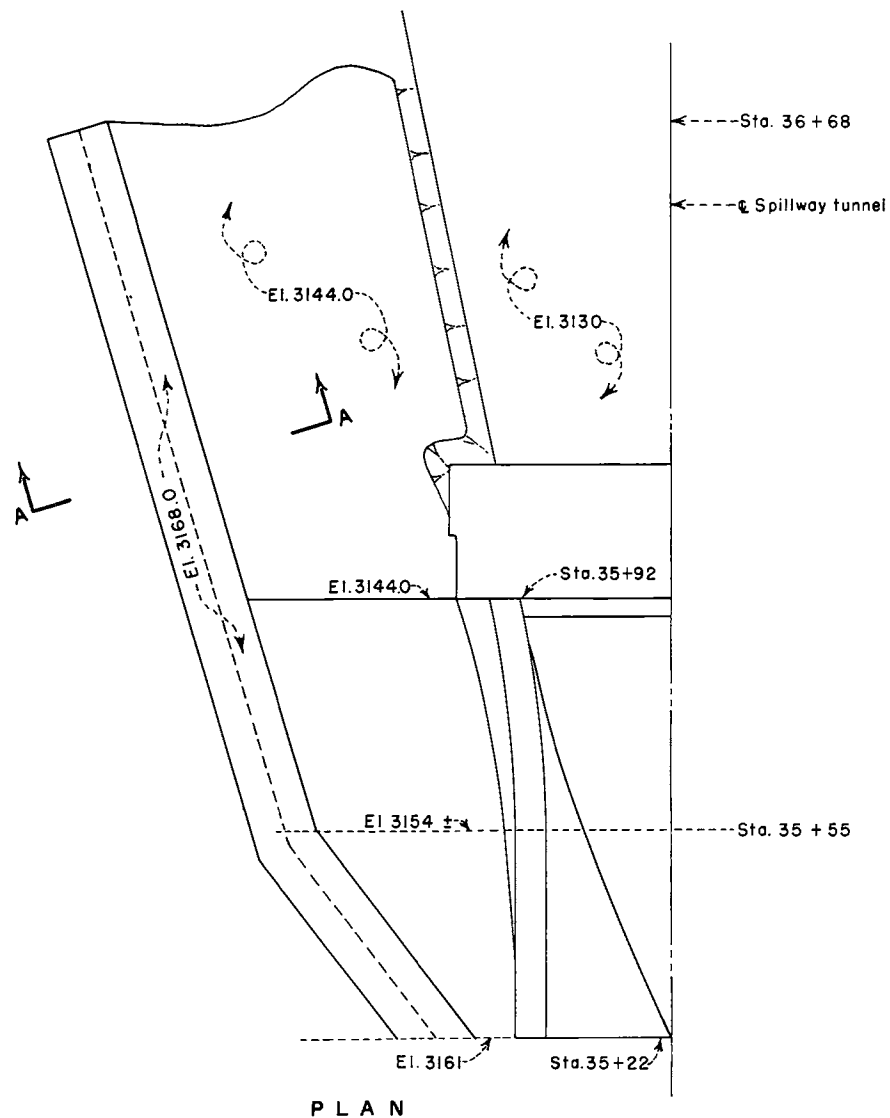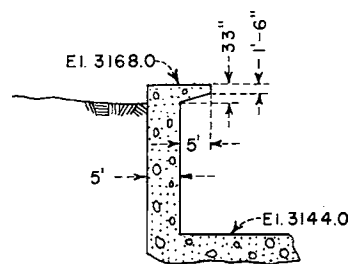
- - - - Sta. 36 + 68

- - - - ℄ Spillway tunnel

El. 3144.0

El. 3130

A

A

El. 3168.0

El. 3144.0

- - - Sta. 35+92

El. 3154 ±

Sta. 35 + 55

El. 3161

Sta. 35+22

PLAN

El. 3168.0

33"

1'-6"

5'

5'

El. 3144.0

SECTION A-A

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
RIGHT DIVERSION TUNNEL—SIDE CHANNEL STUDY
FIRST MODIFICATION

662

Figure 18
Report Hyd-469

50-foot long deflector wall on right
side at tunnel portal. Canyon wall
and riverbed molded in erodible
sand-cement mixture.

Extent of **canyon** wall erosion after
30 **minutes operation** at 50,000 cfs.

Discharge = 50,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Right Diversion Tunnel Deflector Wall Studies

Figure 19
Report Hyd-469

Q = 15,000 cfs   Q = 50,000 cfs

Superelevated extension wall **downstream**
from deflector wall.

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Right Diversion Tunnel Superelevated **Deflector** Wall

FIGURE 20
REPORT HYD. 469
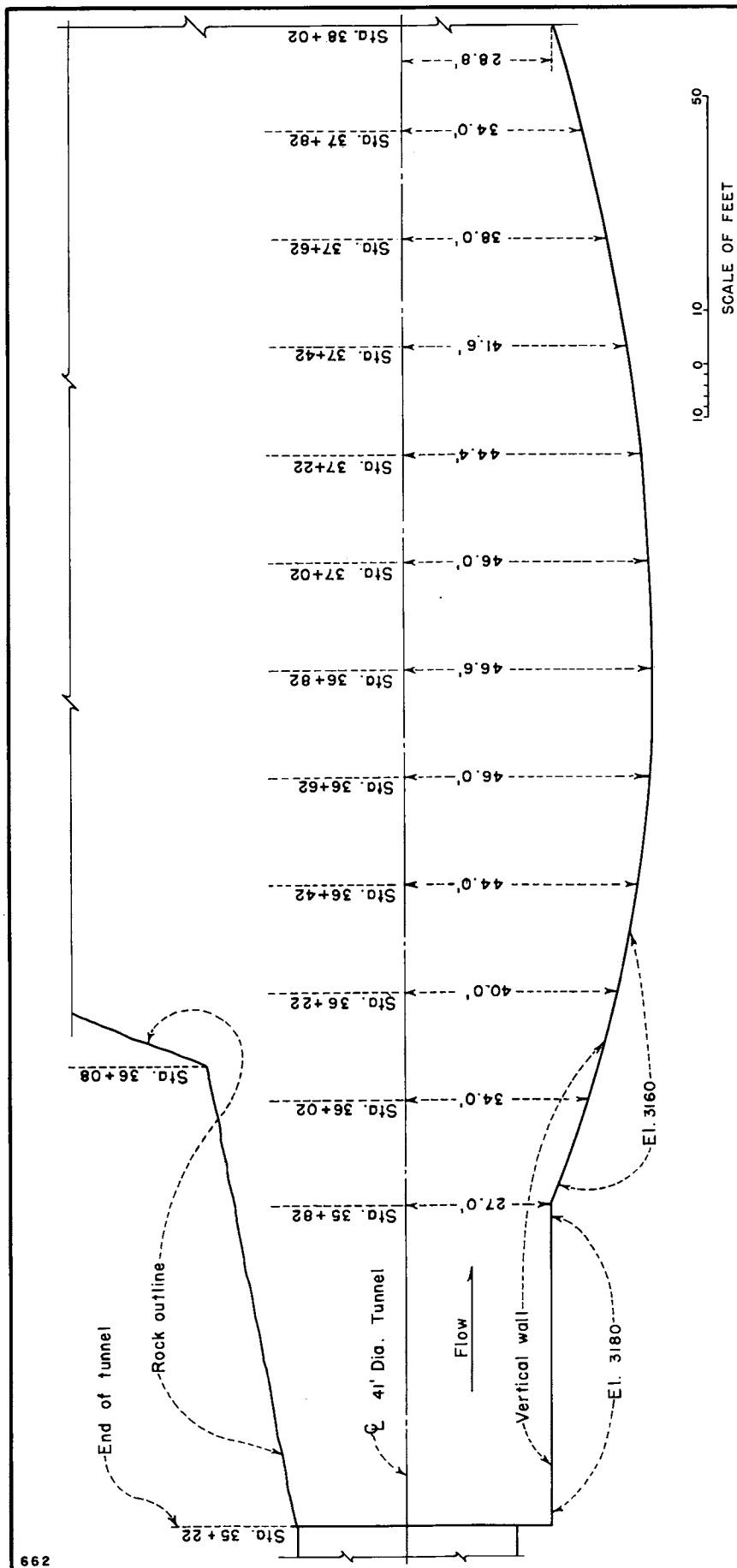
GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
LOW CURVED WALL ON RIGHT SIDE
PORTAL OF RIGHT DIVERSION TUNNEL

SCALE OF FEET

662

Discharge = 15,000 cfs   Discharge = 35,000 cfs   Discharge = 70,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Right Diversion Tunnel
Flow conditions with low curved wall
protecting right canyon wall

Figure 21
Report Hyd-469

Normal flow from tunnel.

Jet deflected by tailwater surging.

Discharge = 50,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Right Diversion Tunnel
Surging action caused by tailwater conditions

Figure 22
Report Hyd-469

Discharge = 35,000 cfs

Normal flow from tunnel.

Jet deflected by tailwater surging.

Discharge = 50,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Right Diversion Tunnel
Flow conditions with low straight wall
protecting right canyon wall
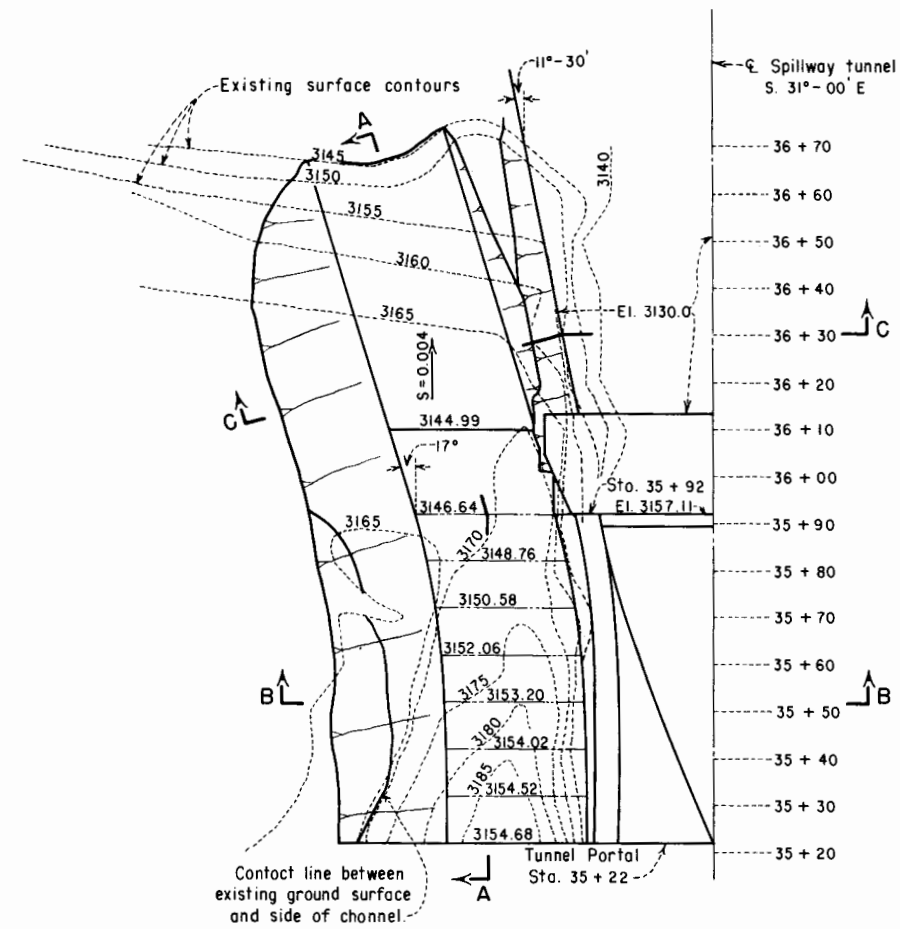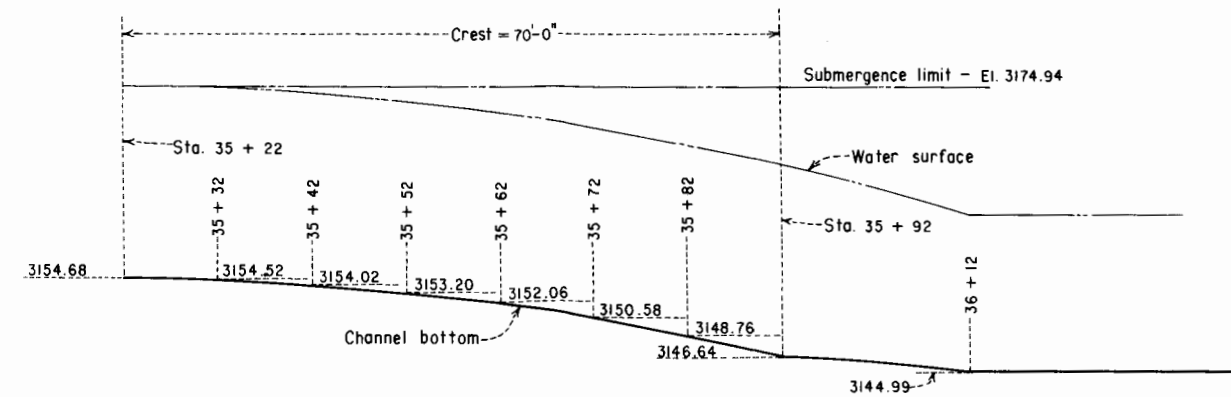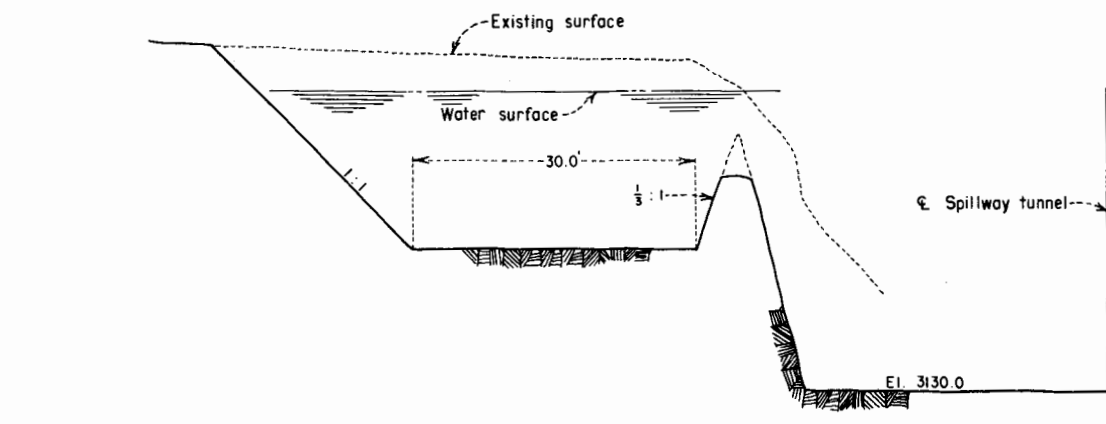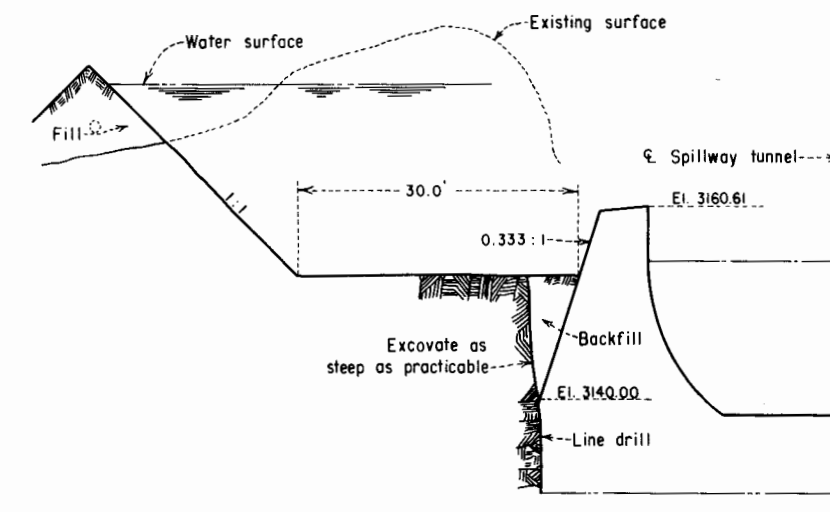
Figure 23
Report Hyd-469

FIGURE 15
REPORT HYD. 469

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
RIGHT DIVERSION TUNNEL- SIDE CHANNEL STUDIES

Figure 25
Report Hyd-469

GLEN CANYON DAM SPILLWAYS

Protective Wall at Right Diversion Tunnel

FIGURE 26
REPORT HYD. 469

PRELIMINARY

1 ST. REVISION

2 ND. REVISION

Tunnel ₵
—Sta. 19 + 86
El. 3635

3 RD. REVISION

4 TH. REVISION

45° Chamfer

VELOCITY DISTRIBUTION IN APPROACH CHANNELS				
LOCA-TION	PRELIMINARY		3 RD. REVISION	
O	V. @ EI. 3697	V. @ EI. 3639	V. @ EI. 3697	V. @ EI. 3639
1	13.4	16.8	14.2	16.6
2	16.0	15.6	15.1	14.6
3	15.0	16.9	17.2	16.2
4	6.8	5.9	13.4	16.3
5	8.8	8.7	11.9	10.5
6	16.7	12.8	15.8	14.3
7	2.6	1.8	15.8	19.8
8	2.6	2.0	5.4	6.9
9	4.7	3.4	6.8	9.8
10	7.7	5.3	9.6	12.4
11	7.1	7.2		

NOTES
1. Dashed lines show preliminary channel
2. Only bottom of slope shown in revisions.
3. Slopes excovated on 0.25:1 slope.
4. Circled points show velocity measuring stations.
5. Velocities are in feet per second.
6. Velocities obtained for Q = 138,000 c.f.s.
   Reservoir elevation 3711

0   50   100   150   200   250
SCALE IN FEET

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
LEFT SPILLWAY APPROACH CHANNEL REVISIONS

662

Figure 27
Report Hyd-469

Discharge = 138,000 cfs
Reservoir Elev. 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Preliminary Left Spillway Approach Channel

Figure 28
Report Hyd-469

A. First Revision

Discharge = 138,000 cfs
Reservoir Elev. 3711.0

B. Second Revision

Discharge = 138,000 cfs
Reservoir Elev. 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Revised Left Spillway Approach Channel

Figure 29
Report Hyd-469

Discharge = 138,000 cfs
Reservoir Elev. = 3711.0

A.   Third Revision

Discharge = 138,000 cfs
Reservoir Elev. = 3711.0

B.   Recommended

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Revised Left Spillway Approach Channel

FIGURE 24
REPORT HYD. 469

PLAN

SECTION A-A

SECTION B-B

NOTE
For general notes and concrete requirements,
see Dwg. 557-D-2271
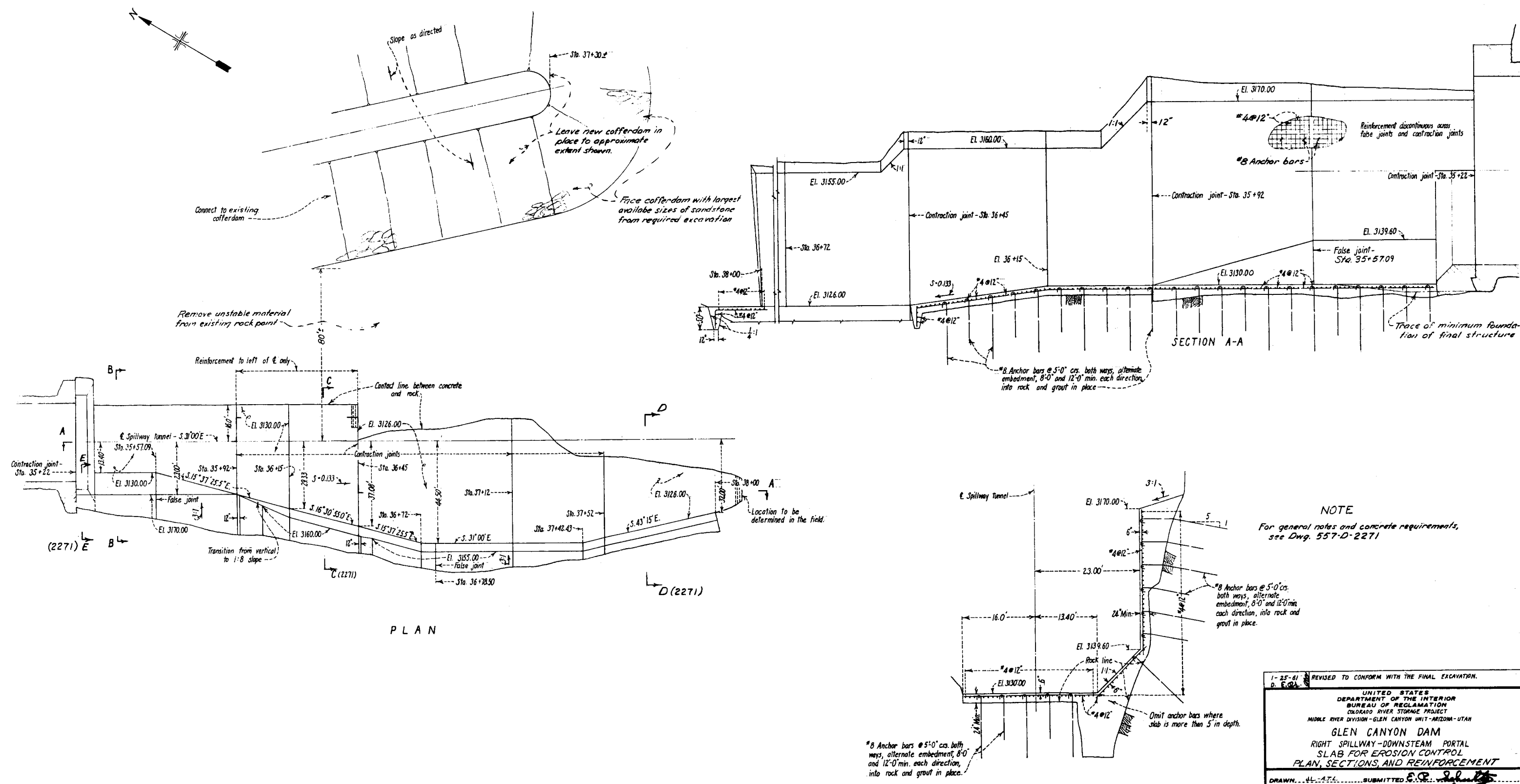
1-25-61 REVISED TO CONFORM WITH THE FINAL EXCAVATION.
D. E.O.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIVISION-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
RIGHT SPILLWAY-DOWNSTEAM PORTAL
SLAB FOR EROSION CONTROL
PLAN, SECTIONS, AND REINFORCEMENT

DRAWN   SUBMITTED
TRACED   RECOMMENDED
CHECKED   APPROVED

DENVER, COLORADO,   NOVEMBER 30, 1960
SHEET 1 OF 2

557-D-2270

Sta. 20 + OO

Sta. 20 + OO

DISTANCE UPSTREAM FROM STATION 20 + OO

0

50

100

150

200

250

300

350

400

14.1/16.2

20.2/16.1

17.6/15.9

12.8/12.0

16.4/14.0

18.5/17.2

13.3/12.1

7.4/8.4

15.8/14.7

6.0/7.4

11.6/10.4

El. 3635.00

9.0/8.2

7.1/8.5

8.1/6.0

7.1/3.9

5.2/6.9

¢

100   50   0   50   100   150   200   250

DISTANCE FROM ¢ OF SPILLWAY

LEFT APPROACH CHANNEL

DISTANCE UPSTREAM FROM STATION 20 + OO

0

50

100

150

200

250

300

350

400

17.0/16.6

15.0/11.3

14.6/12.1

14.0/14.5

8.9/12.4

17.8/14.1

12.7/9.9

8.2/8.8

El. 3635.00

9.5/6.6

6.2/6.3

¢

400   350   300   250   200   150   100   50   0   50   100

DISTANCE FROM ¢ OF SPILLWAY

RIGHT APPROACH CHANNEL

NOTES

1.   o V @ El. 3697 / V @ El. 3639

2. Velocities are in feet per second.

3. Velocities measured for Q = 138,000 c.f.s. per spillway.
    Reservoir elevation 3711.

GLEN CANYON DAM SPILLWAYS

1:63.48 SCALE MODEL

VELOCITY DISTRIBUTION IN RECOMMENDED APPROACH CHANNELS

FIGURE 32
REPORT HYD. 469

—Tunnel ℄

—Sta. 19 + 86

PRELIMINARY

—Tunnel ℄

Sta. 19 + 86 ---

El. 3635---

1ST. REVISION

---Tunnel ℄

--Sta. 19 + 86
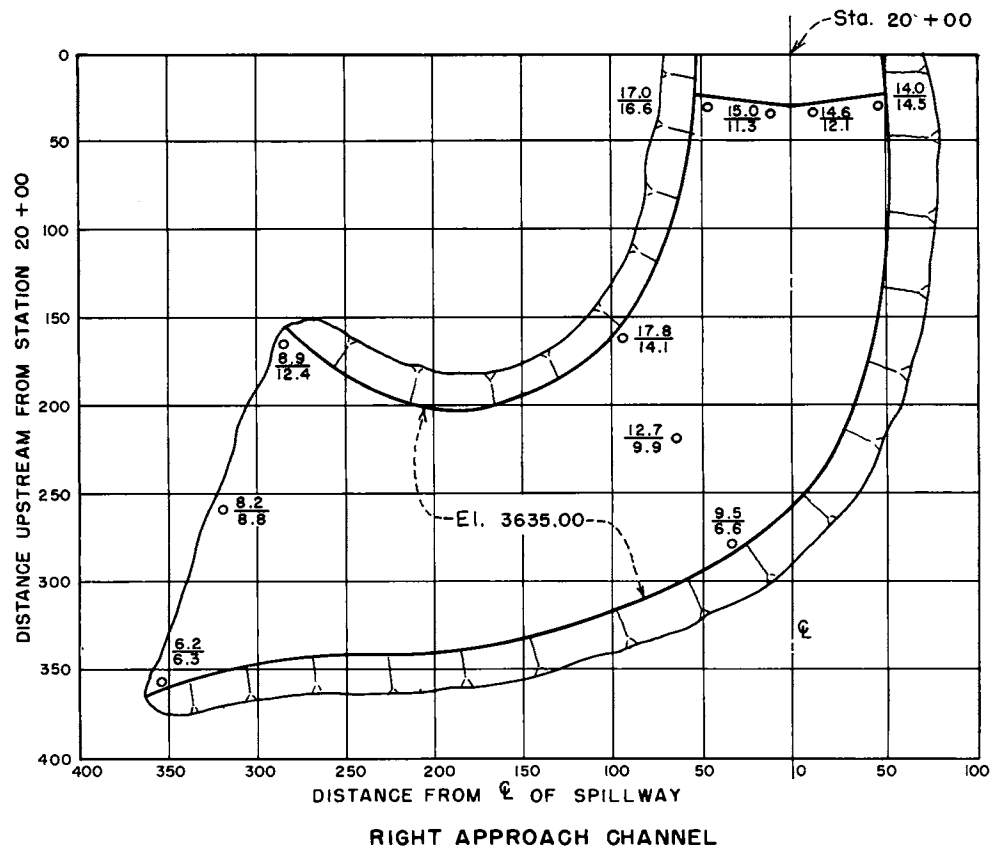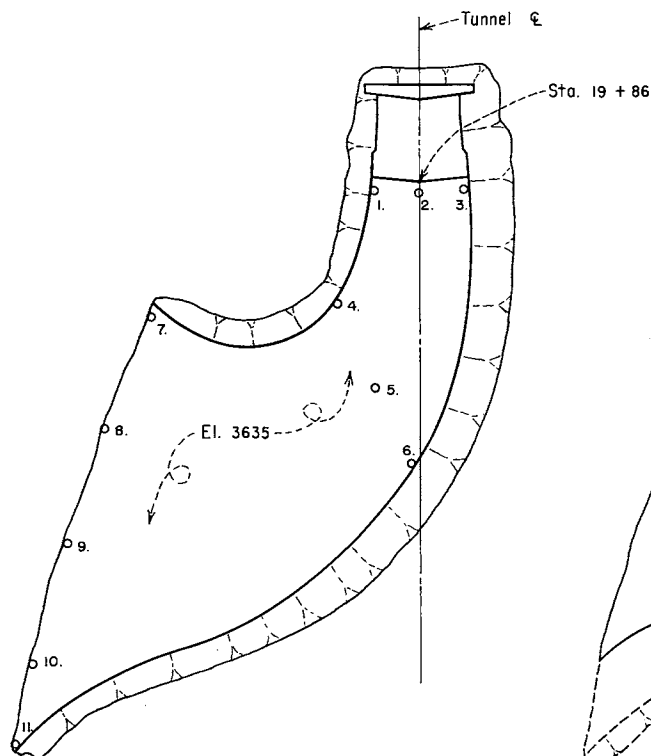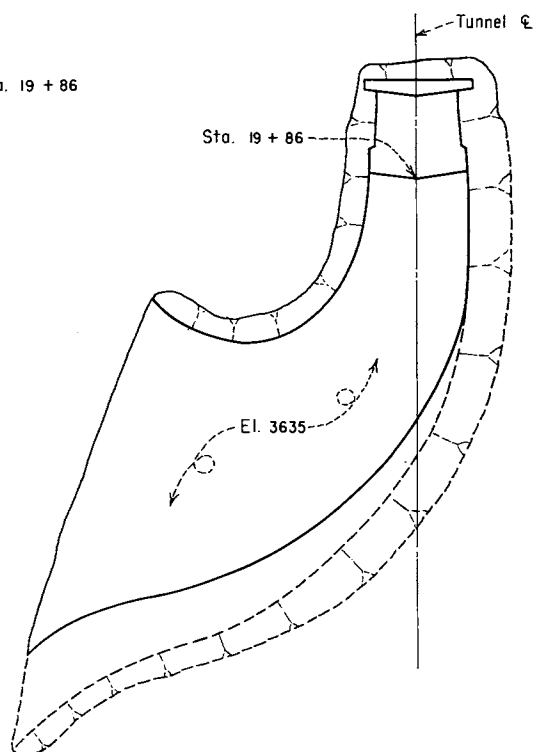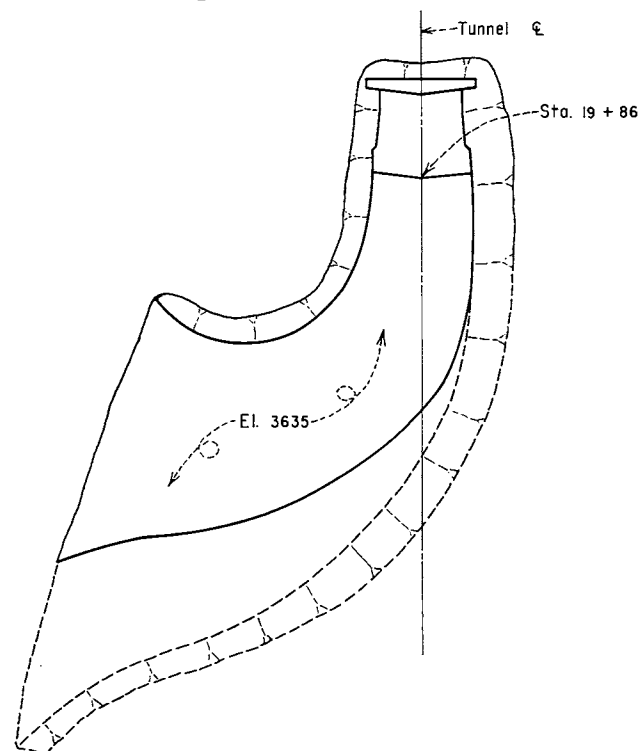
El. 3635---

2 ND. REVISION

VELOCITY DISTRIBUTION IN PRELIMINARY APPROACH		
LOCATION	V @ EL. 3697	V @ EL. 3639
○ 1	16.0	15.6
○ 2	15.4	14.8
○ 3	13.9	15.4
○ 4	14.1	11.4
○ 5	10.0	8.0
○ 6	7.0	6.5
○ 7	4.9	4.0
○ 8	4.4	3.6
○ 9	4.2	3.2
○ 10	3.8	3.1
○ 11	4.2	3.6

NOTES
1. Dashed lines show preliminary channel.
2. Only bottom of slope shown in revisions.
3. Side slopes excavated on 0.25:1 slope.
4. Circled points show velocity measuring stations.
5. Velocities are in feet per second.
6. Velocities obtained for Q = 138,000 c.f.s. Reservoir elevation 3711.

0   50   100   150   200   250
SCALE IN FEET

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
RIGHT SPILLWAY APPROACH CHANNEL REVISIONS

Figure 33
Report Hyd-469

Discharge = 138,000 cfs
Reservoir Elev. 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Preliminary Right Spillway Approach Channel
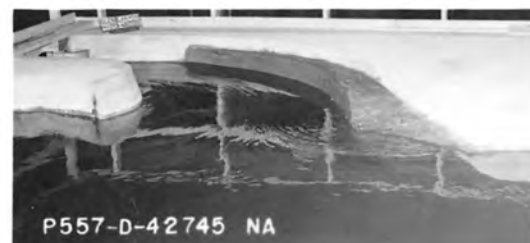
Figure 34
Report Hyd-469

A. First Revision

Discharge = 138,000 cfs
Reservoir Elev. = 3711.0
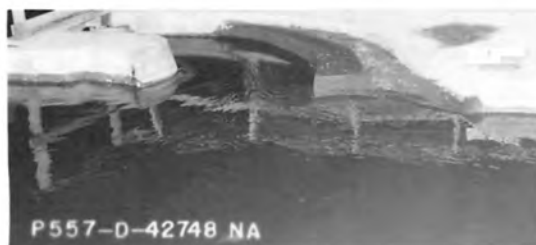
B. Second Revision

Discharge = 138,000 cfs
Reservoir Elev. = 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Revisions to Right Spillway Approach Channel

Figure 35
Report Hyd-469

P557-D-42749 NA

P557-D-42750 NA

P557-D-42751 NA

Discharge = 138,000 cfs
Reservoir Elev. 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Recommended Right Spillway Approach Channel

Figure 36
Report Hyd-469

P557-D-42767-N A

Discharge = 138,000 cfs
Reservoir Elev. = 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow with superelevated floor in
recommended approach channel

FIGURE 30
REPORT HYD. 469

LEFT SPILLWAY APPROACH CHANNEL

RIGHT SPILLWAY APPROACH CHANNEL

SECTION A-A

SECTION B-B

SECTION C-C

SCALE OF FEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
SPILLWAY APPROACH CHANNELS
EXCAVATION

DRAWN........J.L....... SUBMITTED...E.R.Dexter...
TRACED........R.T.S...... RECOMMENDED...O.L.Rice...
CHECKED....L.E.-E.E.E... APPROVED.......L.G.Puls...
AGT. CHIEF DESIGNING ENGINEER

DENVER, COLORADO, JUNE 28, 1957   557-D-442

FIGURE 38
REPORT HYD. 469

ELEVATION ALONG TUNNEL ℄

--- AXIS OF CREST
STA. 20 + 00

TUNNEL PORTAL---
(PIER NOT SHOWN)

---El. 3648.0

El. 3543.47
STA. 21 + 46.39---

El. 3451.27
STA. 22 + 10.9---

PRESSURES (IN FEET OF WATER)

PIEZ #	STATION	ELEV.	PRESSURE	PIEZ #	STATION	ELEV.	PRESSURE
1	19 + 85.6	3641.5	57.5	20	21 + 99.5	3466.7	16.8
2	19 + 90.6	3646.0	33.0	21	20 + 54.2	3653.8	11.3
3	20 + 00.0	3648.0	22.5	22	20 + 70.0	3643.4	6.0
4	20 + 05.5	3647.7	23.1	23	20 + 94.1	3628.6	6.7
5	20 + 11.0	3647.4	20.2	24	21 + 09.8	3616.9	4.9
6	20 + 17.0	3646.5	16.8	25	21 + 17.7	3609.0	5.5
7	20 + 23.0	3645.1	15.4	26	21 + 29.0	3601.0	5.0
8	20 + 29.0	3643.3	14.6	27	21 + 21.1	3590.0	7.3
9	20 + 35.0	3641.1	13.8	28	21 + 49.5	3583.1	2.3
10	20 + 41.0	3638.6	11.2	29	21 + 39.6	3575.1	-0.7
11	20 + 61.0	3628.4	9.3	30	21 + 64.0	3561.4	0.3
12	20 + 79.0	3615.5	7.1	31	21 + 57.5	3556.1	-12.9
13	20 + 93.6	3603.0	-1.7	32	21 + 70.3	3556.7	-9.3
14	21 + 01.5	3599.2	-1.5	33	21 + 64.2	3549.0	-13.7
15	21 + 12.3	3583.4	12.9	34	21 + 77.6	3543.4	1.4
16	21 + 27.5	3565.6	4.2	35	21 + 72.7	3538.4	1.7
17	21 + 42.0	3547.3	1.8	36	21 + 93.6	3510.6	6.2
18	21 + 56.5	3528.1	6.3	37	22 + 15.2	3481.9	14.0
19	21 + 78.0	3497.4	16.2	38	22 + 23.7	3469.9	2.0

NOTES:
1. For details of transition see figure 42.
2. For details of crest see figure 37.
3. Piezometers located in left bay of left spillway nos. 1-20 on invert ℄, nos. 21-38 on left wall.
4. Pressures obtained for discharge of 138,000 c.f.s.

SCALE IN FEET

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRESSURES ON CREST AND PRELIMINARY TRANSITION

FIGURE 38
REPORT HYD. 469

ELEVATION ALONG TUNNEL ℄

PRESSURES (IN FEET OF WATER)

PIEZ #	STATION	ELEV.	PRESSURE	PIEZ #	STATION	ELEV.	PRESSURE
1	19 + 85.6	3641.5	57.5	20	21 + 99.5	3466,7	16.8
2	19 + 90.6	3646.0	33.0	21	20 + 54.2	3653.8	11.3
3	20 + 00.0	3648.0	22.5	22	20 + 70.0	3643.4	6.0
4	20 + 05.5	3647.7	23.1	23	20 + 94.1	3628.6	6.7
5	20 + 11.0	3647.4	20.2	24	21 + 09.8	3616.9	4.9
6	20 + 17.0	3646.5	16.8	25	21 + 17.7	3609.0	5.5
7	20 + 23.0	3645.1	15.4	26	21 + 29.0	3601.0	5.0
8	20 + 29.0	3643.3	14.6	27	21 + 21.1	3590.0	7.3
9	20 + 35.0	3641.1	13.8	28	21 + 49.5	3583.1	2.3
10	20 + 41.0	3638.6	11.2	29	21 + 39.6	3575.1	-0.7
11	20 + 61.0	3628.4	9.3	30	21 + 64.0	3561.4	0.3
12	20 + 79.0	3615.5	7.1	31	21 + 57.5	3556.1	-12.9
13	20 + 93.6	3603.0	-1.7	32	21 + 70.3	3556.7	-9.3
14	21 + 01.5	3599.2	-1.5	33	21 + 64.2	3549.0	-13.7
15	21 + 12.3	3583.4	12.9	34	21 + 77.6	3543.4	1.4
16	21 + 27.5	3565.6	4.2	35	21 + 72.7	3538.4	1.7
17	21 + 42.0	3547.3	1.8	36	21 + 93.6	3510.6	6.2
18	21 + 56.5	3528.1	6.3	37	22 + 15.2	3481.9	14.0
19	21 + 78.0	3497.4	16.2	38	22 + 23.7	3469.9	2.0

NOTES:
1. For details of transition see figure 42.
2. For details of crest see figure 37.
3. Piezometers located in left bay of left spillway nos. 1-20 on invert ℄, nos. 21-38 on left wall.
4. Pressures obtained for discharge of 138,000 c.f.s.

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRESSURES ON CREST AND PRELIMINARY TRANSITION

FIGURE 38
REPORT HYD. 469

ELEVATION ALONG TUNNEL ℄

--- AXIS OF CREST
STA. 20 + 00

TUNNEL PORTAL---
(PIER NOT SHOWN)

---El. 3648.0

El. 3543.47
STA. 21 + 46.39----->

El. 3451.27
STA. 22 + 10.9----->

PRESSURES (IN FEET OF WATER)

PIEZ #	STATION	ELEV.	PRESSURE	PIEZ #	STATION	ELEV.	PRESSURE
1	19 + 85.6	3641.5	57.5	20	21 + 99.5	3466,7	16.8
2	19 + 90.6	3646.0	33.0	21	20 + 54.2	3653.8	11.3
3	20 + 00.0	3648.0	22.5	22	20 + 70.0	3643.4	6.0
4	20 + 05.5	3647.7	23.1	23	20 + 94.1	3628.6	6.7
5	20 + 11.0	3647.4	20.2	24	21 + 09.8	3616.9	4.9
6	20 + 17.0	3646.5	16.8	25	21 + 17.7	3609.0	5.5
7	20 + 23.0	3645.1	15.4	26	21 + 29.0	3601.0	5.0
8	20 + 29.0	3643.3	14.6	27	21 + 21.1	3590.0	7.3
9	20 + 35.0	3641.1	13.8	28	21 + 49.5	3583.1	2.3
10	20 + 41.0	3638.6	11.2	29	21 + 39.6	3575.1	-0.7
11	20 + 61.0	3628.4	9.3	30	21 + 64.0	3561.4	0.3
12	20 + 79.0	3615.5	7.1	31	21 + 57.5	3556.1	-12.9
13	20 + 93.6	3603.0	-1.7	32	21 + 70.3	3556.7	-9.3
14	21 + 01.5	3599.2	-1.5	33	21 + 64.2	3549.0	-13.7
15	21 + 12.3	3583.4	12.9	34	21 + 77.6	3543.4	1.4
16	21 + 27.5	3565.6	4.2	35	21 + 72.7	3538.4	1.7
17	21 + 42.0	3547.3	1.8	36	21 + 93.6	3510.6	6.2
18	21 + 56.5	3528.1	6.3	37	22 + 15.2	3481.9	14.0
19	21 + 78.0	3497.4	16.2	38	22 + 23.7	3469.9	2.0

NOTES:
1. For details of transition see figure 42.
2. For details of crest see figure 37.
3. Piezometers located in left bay of left spillway nos. 1-20 on invert ℄, nos. 21-38 on left wall.
4. Pressures obtained for discharge of 138,000 c.f.s.

10   0   10   20   30   40   80
SCALE IN FEET

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRESSURES ON CREST AND PRELIMINARY TRANSITION

662

FIGURE 37
REPORT HYD. 469

PLAN
HOIST BRIDGES NOT SHOWN

SECTION H-H

SECTION A-A

SECTION B-B

DETAIL OF "B" GROUT HOLE

SECTION E-E

SECTION C-C

SECTION F-F

SECTION L-L (601)

SECTION J-J (601)

SECTION G-G

SECTION K-K (601)

DETAIL OF HANDRAIL POST RECESS

SECTION D-D

PLAN OF "B" HOLE GROUTING

PLAN OF FLOOR DRAINS

NOTE
For general notes and concrete requirements, see Dwg. 557-D-601

COEFFICIENTS FOR CREST EQUATIONS

No.	A	B	C
1	0.00010645	0.010252	0.11039
2	0.000089840	0.0094979	0.11039
3	0.000074742	0.0087899	0.11039
4	0.000061014	0.0081244	0.11039
5	0.000048525	0.0074983	0.11039
6	0.000037158	0.0069089	0.11039
7	0.000026807	0.0063536	0.11039
8	0.000017380	0.0058301	0.11039
9	0.0000087919	0.0053362	0.11039

12-11-61		REVISED AND ADDED NOTE PERTAINING TO CHANGES AT TOP OF RIGHT END PIER OF LEFT SPILLWAY.
8-26-60		REVISED BLOCKOUTS FOR GATE ANCHORAGE.
10-9-59		REVISED DWG. NUMBERS ON PLAN.
3-10-59		REVISED CURB AND HANDRAIL POST RECESS LOCATION AT TOP OF PIERS. ADDED NOTE PERTAINING TO DUCT BANK.
8-4-58		TRACED. ADDED ACCESS SHAFT TO, AND REVISED DRAINS FOR COUNTERWEIGHT WELLS. REVISED TOPS OF PIERS.

THIS DRAWING SUPERSEDES DWGS. 557-D-86, 87, AND 88 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV- GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
SPILLWAYS
INTAKE STRUCTURES

DRAWN	J.L.	SUBMITTED	E.R. Schultz
TRACED	W.A.P.	RECOMMENDED	O.L. Rice
CHECKED	P.E.O.-M.F.	APPROVED	K.B. Keener

DENVER, COLORADO, FEB. 24, 1958
SHEET 2 OF 2

557-D-602

FIGURE 39
REPORT HYD. 469

GATE OPENING - FEET

DISCHARGE CURVE (Free Crest)

RESERVOIR ELEVATION - FEET

DISCHARGE IN THOUSANDS OF SECOND FEET

NOTES

Discharge curves are for one 40'X 52.5' radial gate.
Gate opening is measured vertically above spillway crest. (El. 3648.00)
The gates in each spillway should be operated simultaneously at equal openings.
Total number of gates-four, two on each spillway.
The curves were obtained from a 1:63.5 scale hydraulic model.

SPILLWAY INTAKE

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV-GLEN CANYON UNIT--ARIZONA-UTAH
GLEN CANYON DAM
SPILLWAY DISCHARGE CURVES
FOR ONE 40'X 52.5' RADIAL GATE

557-D-1386

Flow appearance in transition   No separation   Flow separated from wall

Flow appearance downstream from transition

Discharge = 138,000 cfs, Reservoir Elev. 3711.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow Conditions in Preliminary Transition

Figure 41
Report Hyd-469

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRELIMINARY SPILLWAY TUNNELS

FIGURE 40
REPORT HYD. 469

Figure 43
Report Hyd-469
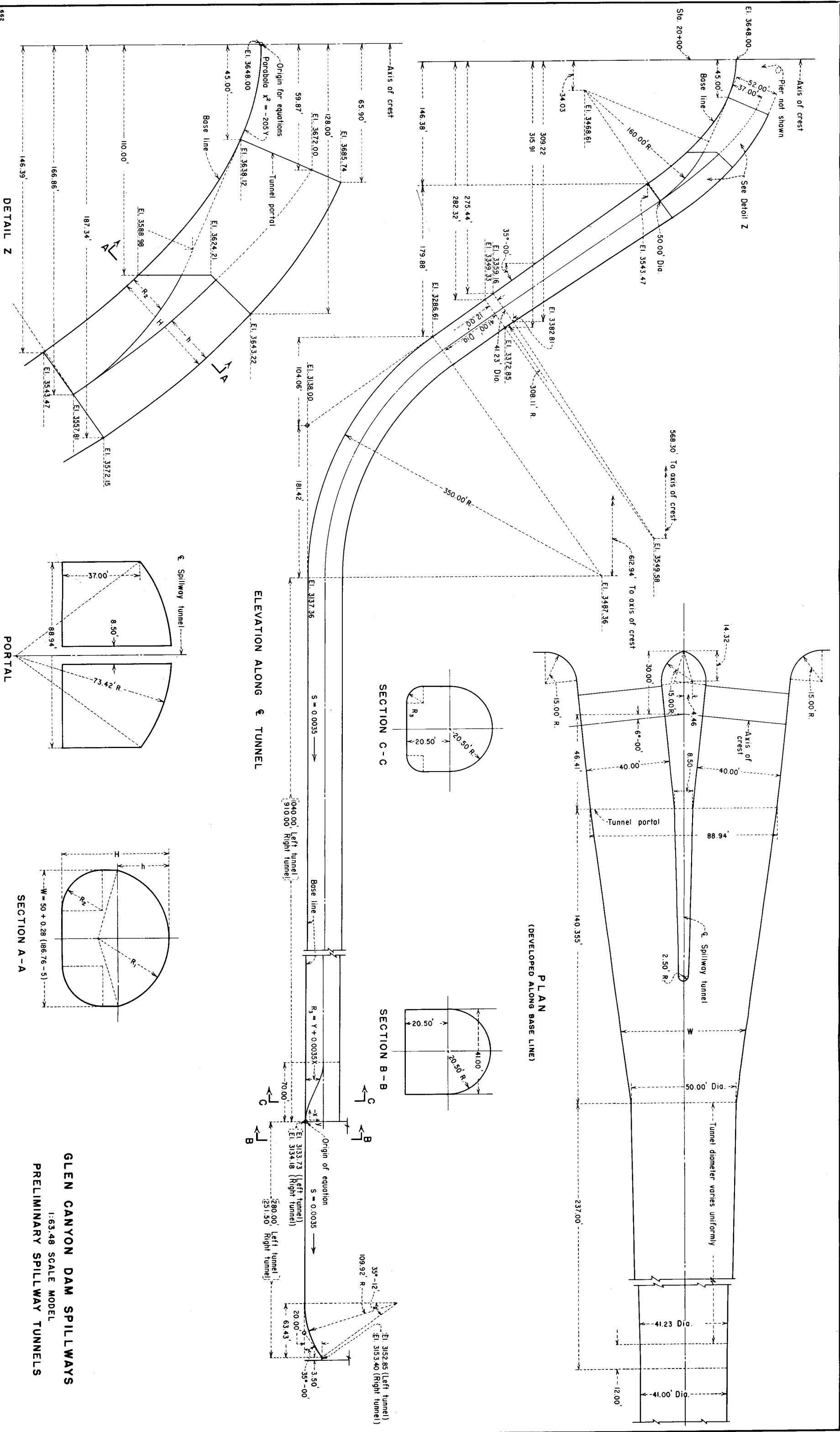
Flow in transition.   Note small surface fin.
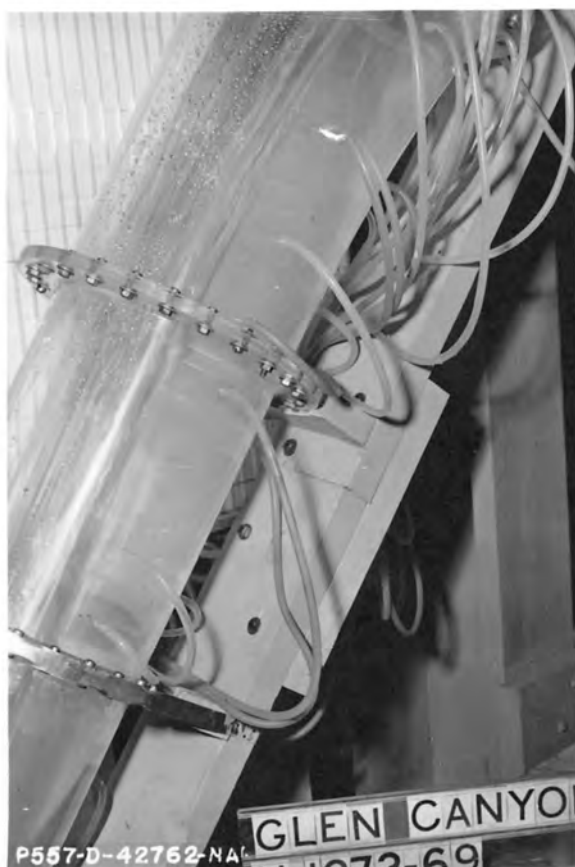
No separation.   Occasional separation from wall.

Flow Downstream from Transition

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Recommended Transition

--AXIS OF CREST
STA. 20 + 00.0

TUNNEL PORTAL----
(PIER NOT SHOWN)

El. 3648.0

NOTES:

1. For details of transition see figure 42.
2. For details of crest see figure 37.
3. Piezometers located in left bay of left spillway nos. 1-21 on invert ℄, nos. 22-48 on left wall.
4. Pressures were taken with reservoir at top of gates or elevation 3711.0, whichever was lowest.

SCALE IN FEET

ELEVATION ALONG TUNNEL ℄

El. 3506.68

STA. 21 + 72.15---

El. 3451.27

STA. 22 + 10.9----

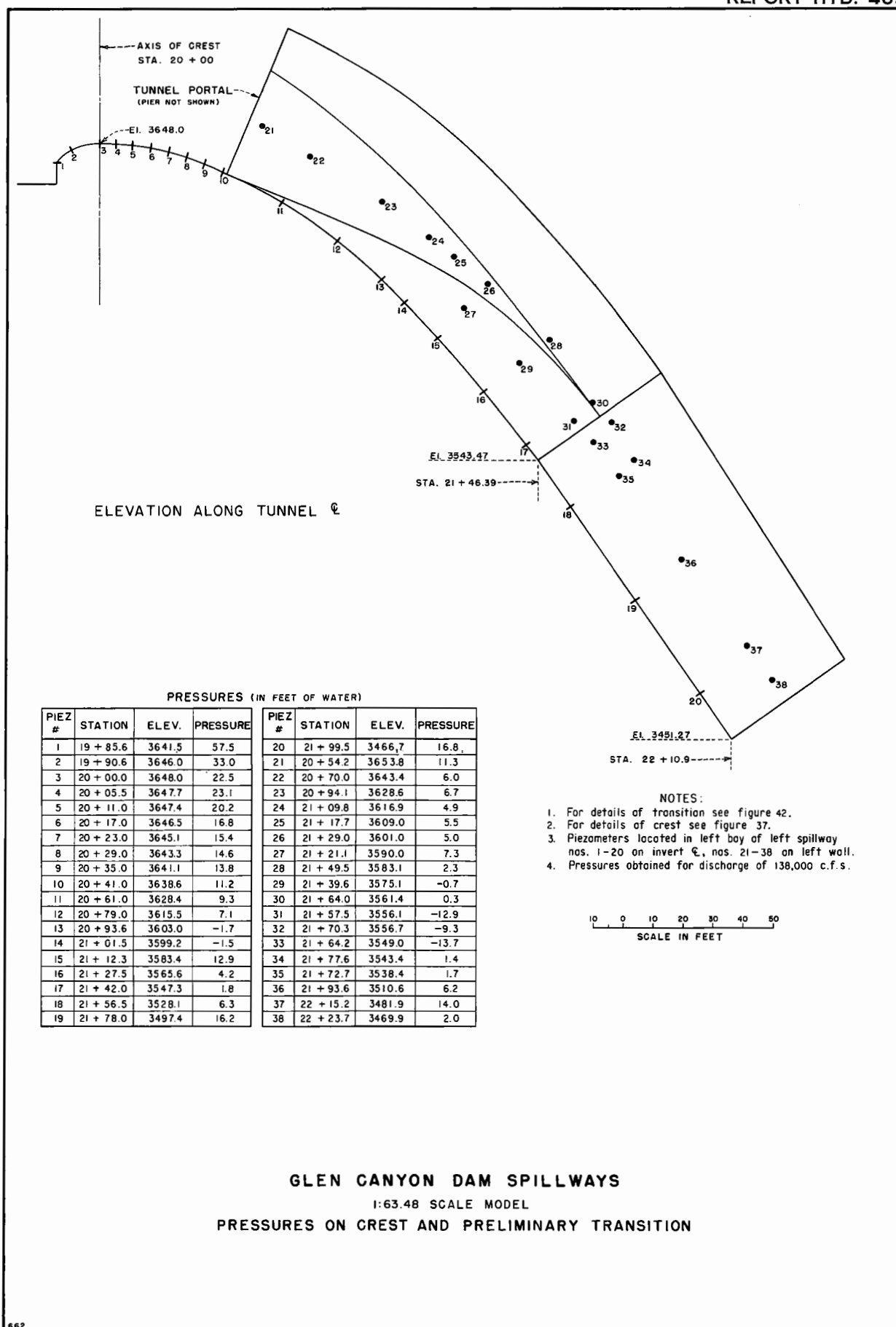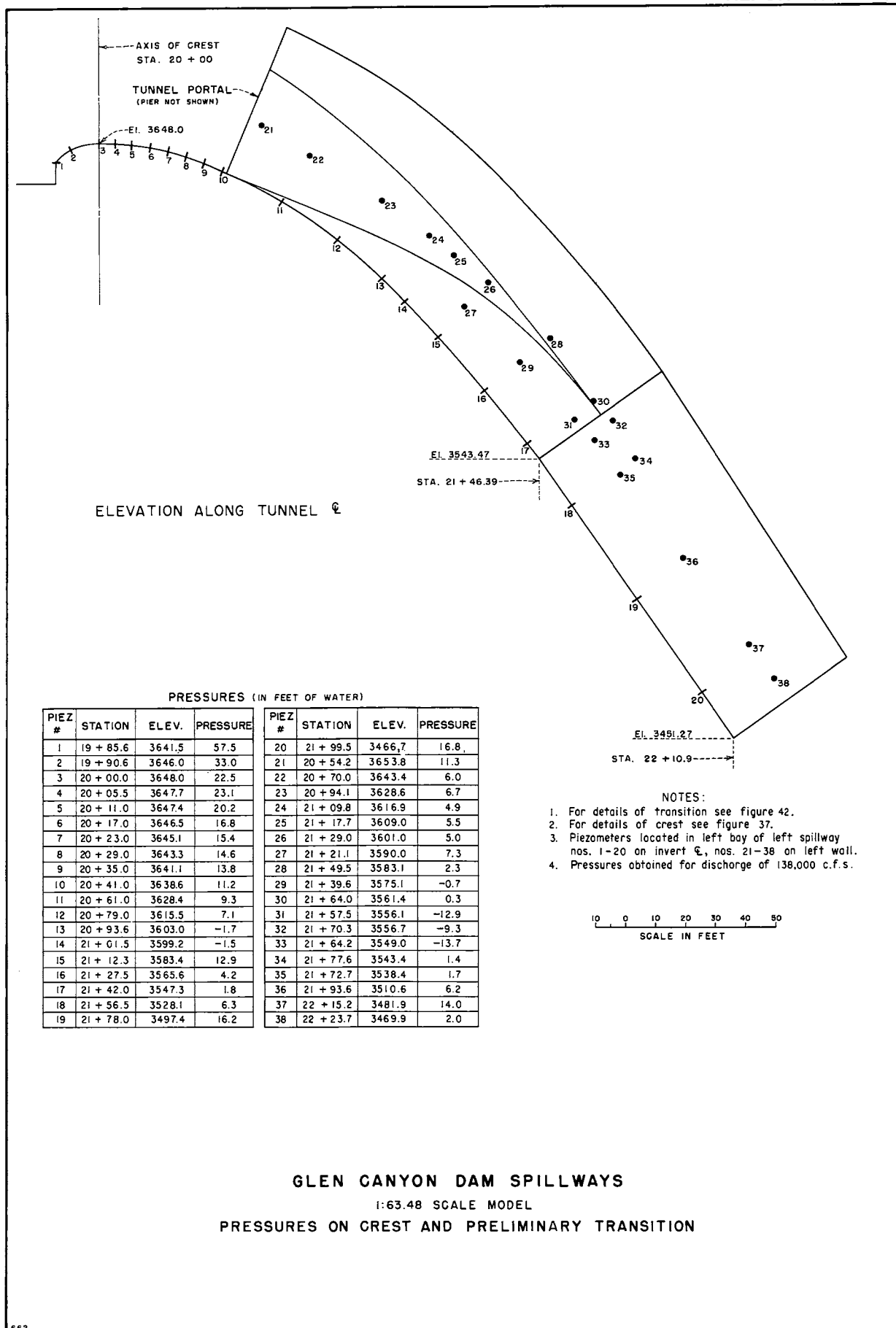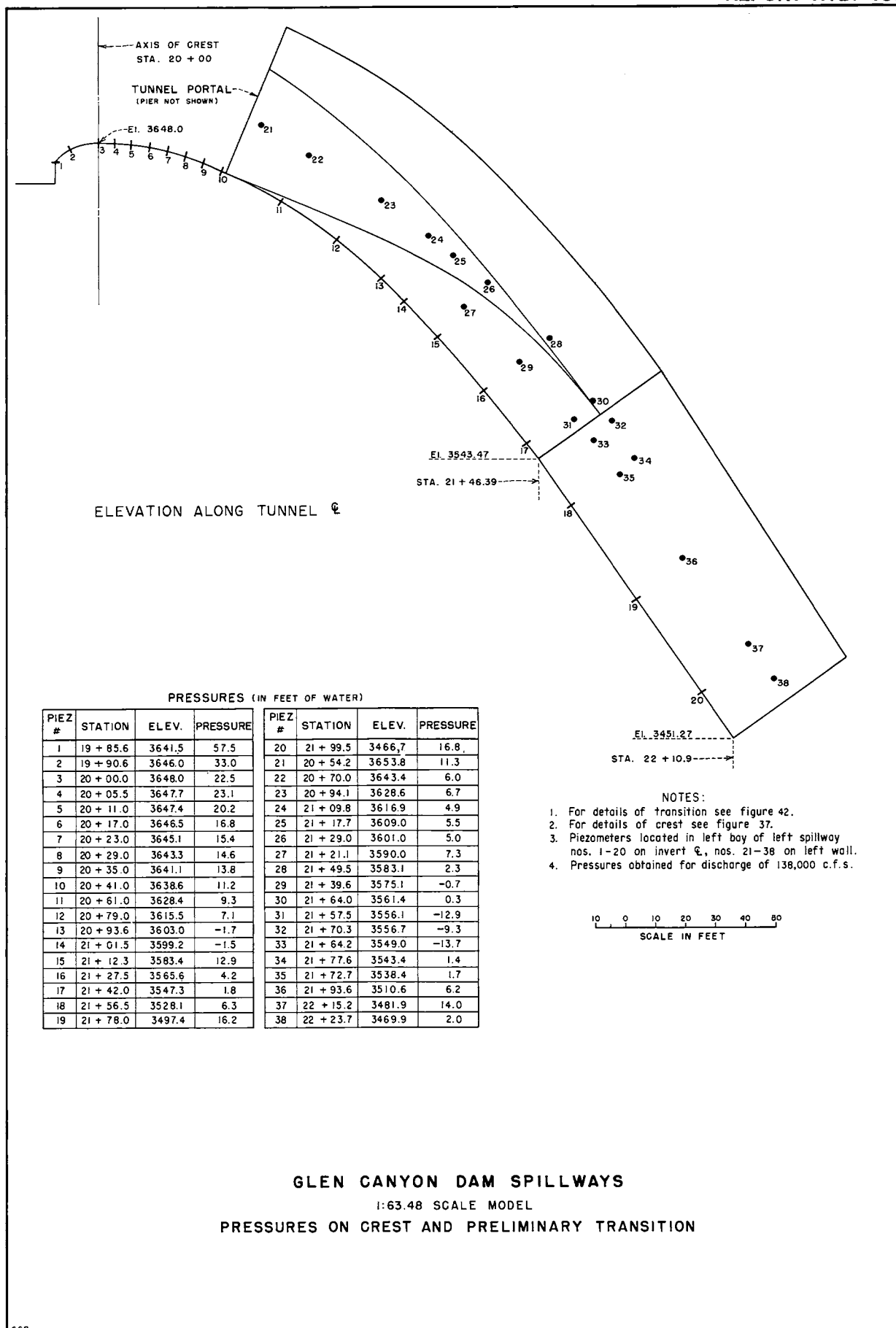GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRESSURES ON CREST AND RECOMMENDED TRANSITION

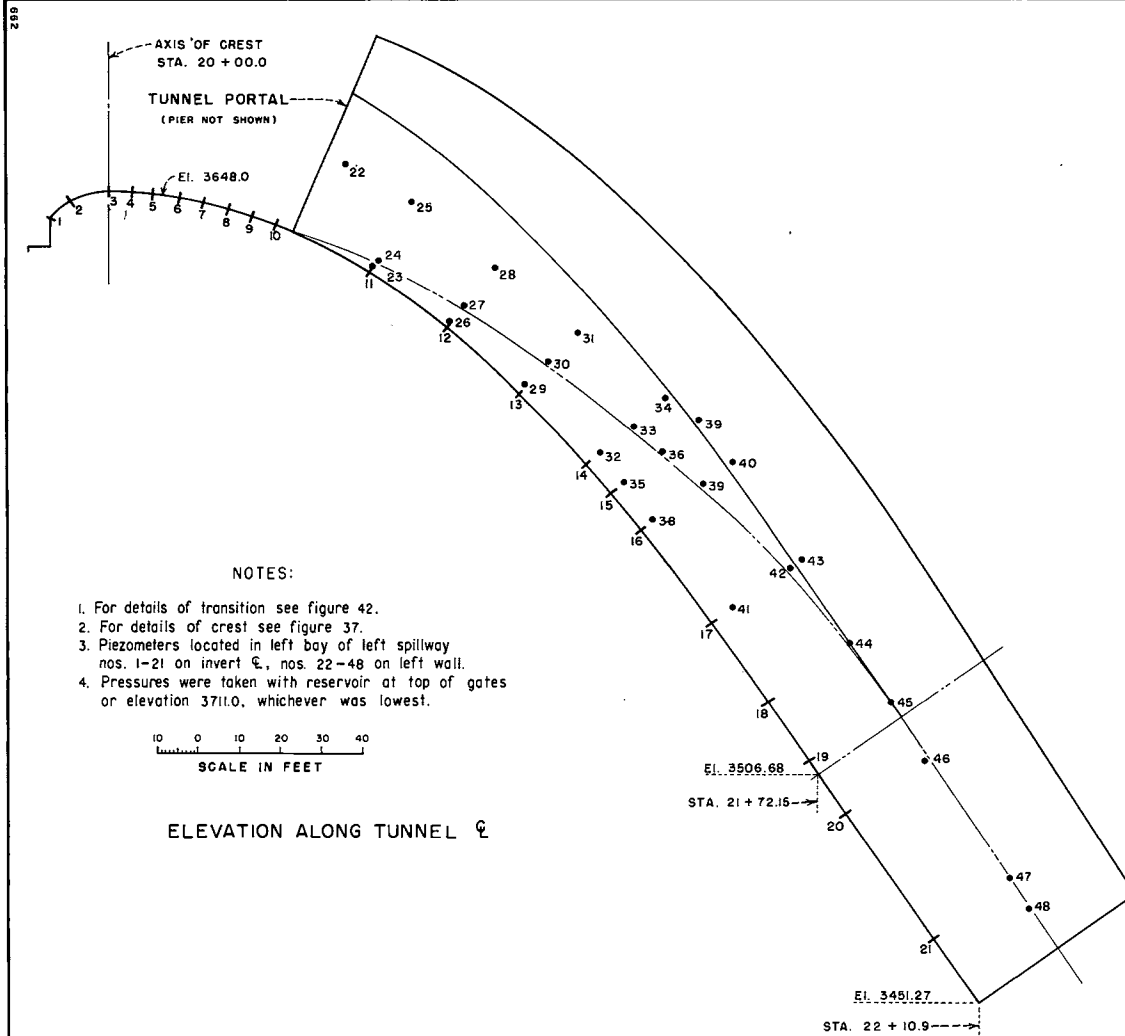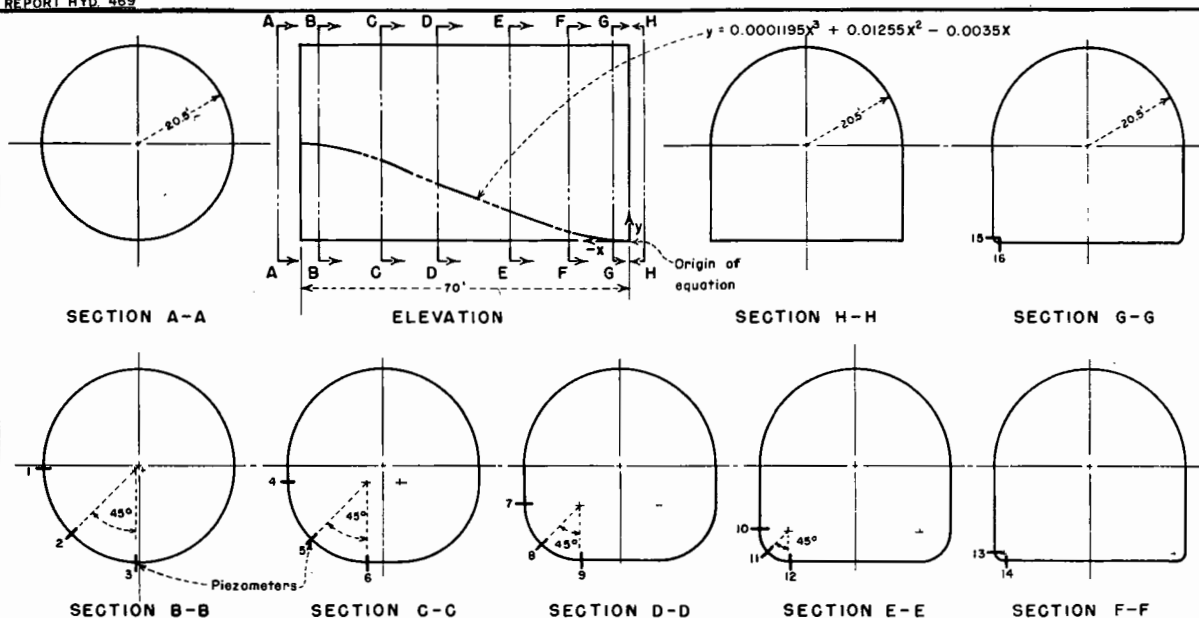| PIEZ	STATION	ELEV.	PRESSURES AT VARIOUS GATE OPENINGS										
			5'	10'	15'	20'	25'	30'	35'	40'	45'	50'	Full
1	19+85.6	3641.5	67.7	68.4	69.2	66.7	64.3	62.6	66.1	61.4	61.3	61.3	57.5
2	19+90.6	3646.0	61.5	52.7	53.5	47.4	44.6	39.9	38.3	38.2	37.3	36.2	33.0
3	20+00.0	3648.0	49.7	35.7	29.5	25.0	23.2	22.7	23.1	23.7	24.2	24.6	22.5
4	20+05.5	3647.7	27.9	19.4	17.8	16.8	17.4	18.5	20.1	21.2	22.7	23.4	23.1
5	20+11.0	3647.4	4.5	6.4	7.3	9.0	10.7	12.8	14.8	16.4	16.7	20.0	20.2
6	20+17.0	3646.5	-0.7	0.4	0.7	2.4	4.5	7.1	8.6	11.4	13.6	16.0	16.8
7	20+23.0	3645.1	-0.3	-0.2	-0.7	0.7	2.8	5.2	6.2	8.9	11.3	14.2	15.4
8	20+29.0	3643.3	-0.6	0.7	0.1	1.2	2.7	5.1	5.7	8.1	10.4	13.5	14.6
9	20+35.0	3641.1	1.5	1.9	1.8	2.7	3.8	5.5	6.4	8.3	10.3	12.9	14.0
10	20+41.0	3638.6	1.7	2.0	2.3	3.1	4.3	5.8	6.8	8.4	10.4	12.4	11.8
11	20+63.2	3628.2	0.9	0.7	0.7	1.1	1.8	2.4	2.8	3.8	4.7	6.3	8.5
12	20+82.0	3614.7	1.9	1.9	1.9	2.2	2.5	2.8	2.7	3.6	4.0	4.8	5.9
13	21+00.0	3599.2	2.3	-0.3	0.2	0	0.5	2.6	2.0	-0.6	0.2	0.5	1.4
14	21+16.0	3581.7	2.4	3.6	3.9	4.4	4.9	5.9	6.2	6.6	7.5	9.9	12.8
15	31+22.0	3574.7	-0.2	0.5	0.7	1.7	2.7	3.6	4.2	5.7	7.6	9.8	9.3
16	21+29.2	3565.6	1.7	2.3	2.5	4.3	5.8	8.1	7.5	8.7	8.8	9.7	8.4
17	21+46.2	3543.4	2.4	3.9	4.6	5.1	6.0	7.3	6.9	7.2	7.4	7.5	8.3
18	21+59.7	3524.0	3.3	6.2	7.5	8.4	8.7	9.7	9.3	9.7	9.7	7.0	11.0
19	21+69.5	3509.9	4.0	5.5	6.6	7.8	8.4	8.7	9.1	9.4	9.7	9.8	9.8
20	21+78.5	3497.4	2.1	4.6	6.6	7.8	9.1	9.9	10.5	11.3	11.3	12.3	10.9
21	21+99.6	3466.9	3.3	5.3	7.3	9.2	10.7	11.5	12.2	13.3	14.5	15.0	16.0
22	20+57.8	3654.3	–	–	–	–	–	–	3.2	1.0	5.4	6.3	9.8
23	20+64.2	3628.8	2.1	2.4	3.5	4.3	5.4	6.3	7.3	8.2	9.2	11.3	12.7
24	20+65.6	3630.3	2.4	2.4	3.4	4.2	5.3	6.5	7.2	8.0	9.1	11.0	12.6
25	20+73.9	3645.8	–	–	–	–	–	–	-1.5	-7.6	-6.0	–	5.8
26	20+83.0	3615.9	3.0	3.6	35	4.0	4.2	5.1	5.3	6.0	6.5	7.9	8.9
27	20+86.5	3619.4	1.5	1.8	2.1	2.2	2.0	3.1	3.7	4.8	4.9	6.2	7.5
28	20+94.0	3629.4	–	–	–	–	0	-7.5	-6.4	-6.8	-6.3	-4.3	4.9
29	20+01.4	3601.1	2.7	3.6	3.6	4.0	4.4	5.2	5.5	5.6	6.3	7.0	7.2
30	21+07.0	3606.0	1.4	2.4	2.3	3.4	39	4.6	4.9	5.3	6.3	69	7.2
31	21+13.8	3613.6	–	–	–	–	1.2	2.7	2.2	3.0	3.6	5.1	5.8
32	21+19.4	3584.3	5.3	7.6	8.0	8.3	8.2	9.0	9.3	9.6	9.8	11.0	11.3
33	21+27.2	3590.8	2.9	3.1	3.8	4.3	4.6	5.2	5.0	5.2	6.3	6.7	6.0
34	21+35.1	3598.0	–	–	–	–	–	1.8	1.8	2.4	2.7	3.9	2.1
35	21+25.6	3577.5	1.7	0	0.8	0.9	1.2	1.9	2.4	2.7	3.4	4.2	4.9
36	21+34.4	3584.6	1.4	2.1	1.9	1.9	1.6	2.3	0.8	2.1	3.0	3.3	2.7
37	21+43.2	3592.1	–	–	–	–	–	2.1	2.0	1.3	2.3	3.4	0.6
38	21+32.0	3568.7	3.5	4.2	4.3	4.7	5.0	5.6	5.9	6.2	7.0	7.8	7.8
39	21+44.3	3576.5	-0.4	1.8	1.6	2.2	2.2	2.8	2.8	3.3	4.0	4.5	4.8
40	21+51.4	3582.7	–	–	–	–	–	1.1	0.9	1.5	1.8	2.4	1.8
41	21+51.5	3547.1	0.3	-0.2	-1.6	-2.0	-2.1	-2.0	-1.9	-1.7	-1.4	-2.1	-1.0
42	21+65.2	3556.3	–	–	–	–	–	0.6	-0.2	0	-0.2	-0.2	-0.1
43	21+68.0	3558.0	–	–	–	–	–	0.7	0.7	0.1	0.3	1.3	0.4
44	21+79.3	3538.0	–	–	–	–	–	1.5	-0.4	1.3	1.6	1.1	0.1
45	21+89.5	3523.8	–	–	–	–	–	2.3	1.9	2.3	2.3	2.7	1.9
46	21+97.5	3510.6	–	–	–	1.3	1.3	1.8	1.9	2.0	2.3	2.9	2.4
47	22+18.2	3481.1	–	–	–	0.8	1.6	2.8	3.4	4.6	35	5.9	6.2
48	22+22.8	3474.9	–	–	–	-2.2	-1.6	-1.8	-1.0	0.7	0.6	2.0	1.6

FIGURE 44
REPORT HYD. 469

FIGURE 45
REPORT HYD 469

$y = 0.0001195x^3 + 0.01255x^2 - 0.0035x$

SECTION A-A   ELEVATION   SECTION H-H   SECTION G-G

SECTION B-B   SECTION C-C   SECTION D-D   SECTION E-E   SECTION F-F

PIEZ NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PRESSURE	7.5	-24.7	15.2	-4.4	-14.5	15.1	13.9	26.9	40.5	52.6	66.7	64.9	69.9	71.2	64.7	63.7

A. PRELIMINARY

SCALE OF FEET

NOTE
Pressure taken at maximum
discharge, 138,000 cfs.

$y = -0.00205x^2 - 0.4135x$
$R = y + 0.0035x$

SECTION A-A   ELEVATION   SECTION G-G

SECTION B-B   SECTION C-C   SECTION D-D   SECTION E-E   SECTION F-F
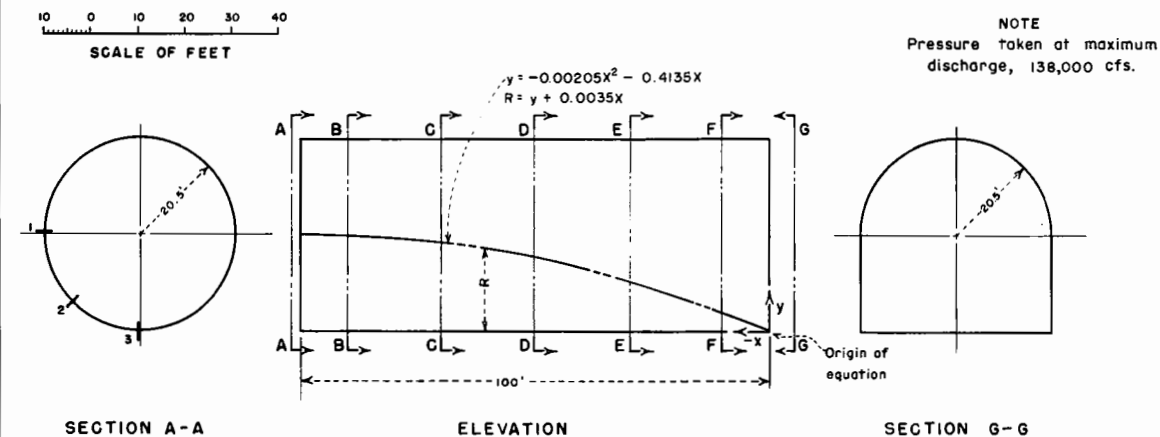
PIEZ NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
PRESSURE	3.8	8.2	22.0	5.5	12.8	16.6	5.7	9.5	18.2	8.0	14.2	28.1	23.6	33.7	41.4	63.6	67.8	68.4

B. REVISED

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
TRANSITION AT DOWNSTREAM TUNNEL PORTAL
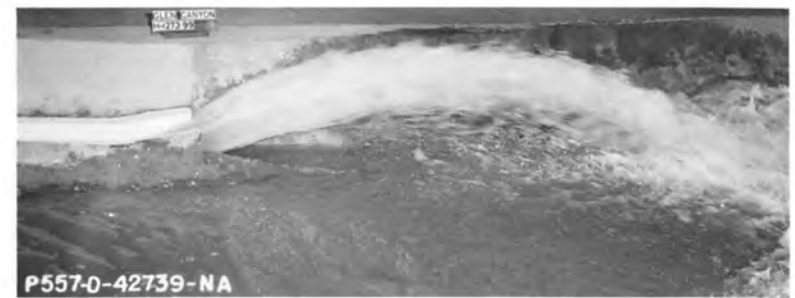
662

Discharge = 25,000 cfs   Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Preliminary Left Flip Bucket

Figure 46
Report Hyd-469

Discharge = 25,000 cfs   Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Preliminary Right Flip Bucket

Figure 47
Report Hyd-469

Figure 48
Report Hyd-469

Discharge = 276,000 cfs
T. W. Elev. = 3180.0

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow Conditions with Both Preliminary Flip Buckets

PLAN

SECTION A-A   SECTION B-B

R = 109.92'
Δ = 35° 12'

TUNNEL PORTAL

S = 0.0035

SECTION C-C

|← 41.0' →|

|← - - - - 217.0' ± - - - - →|← - - 63.43' - - →|← 3.50'

SECTION D-D
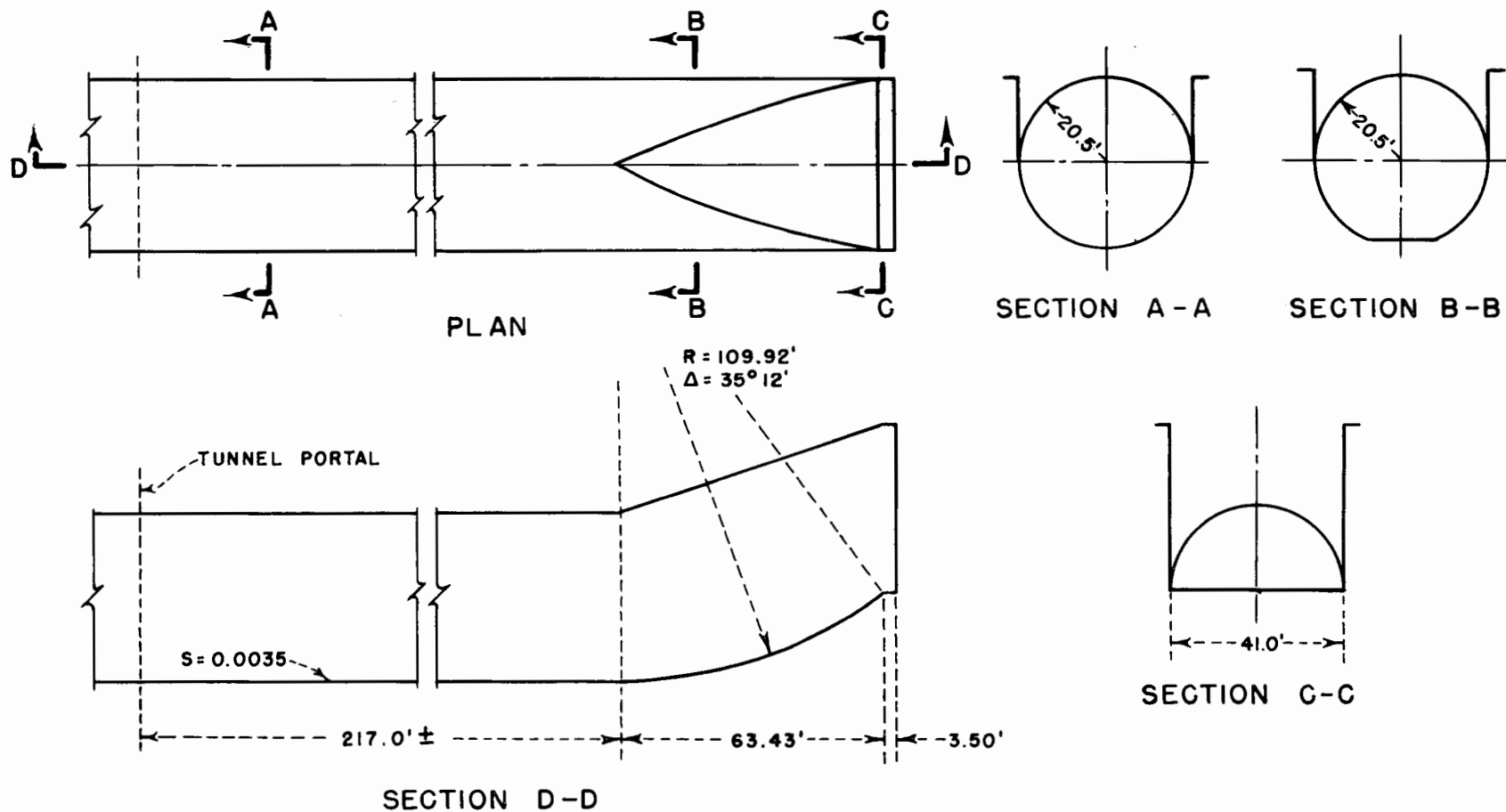
GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
FLIP BUCKETS—FIRST REVISION
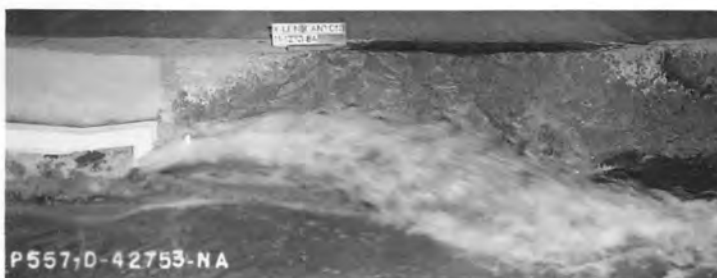
FIGURE 49
REPORT HYD. 469

Discharge = 25,000 cfs

Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Left Flip Bucket - First Revision

Figure 50
Report Hyd-469

Discharge = 25,000 cfs

Discharge = 138,000 cfs
Jet intermittently depressed by
tailwater surges.

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Left Flip Bucket - Second Revision

Figure 51
Report Hyd-469

Discharge = 25,000 cfs

Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Right Flip Bucket - Second Revision
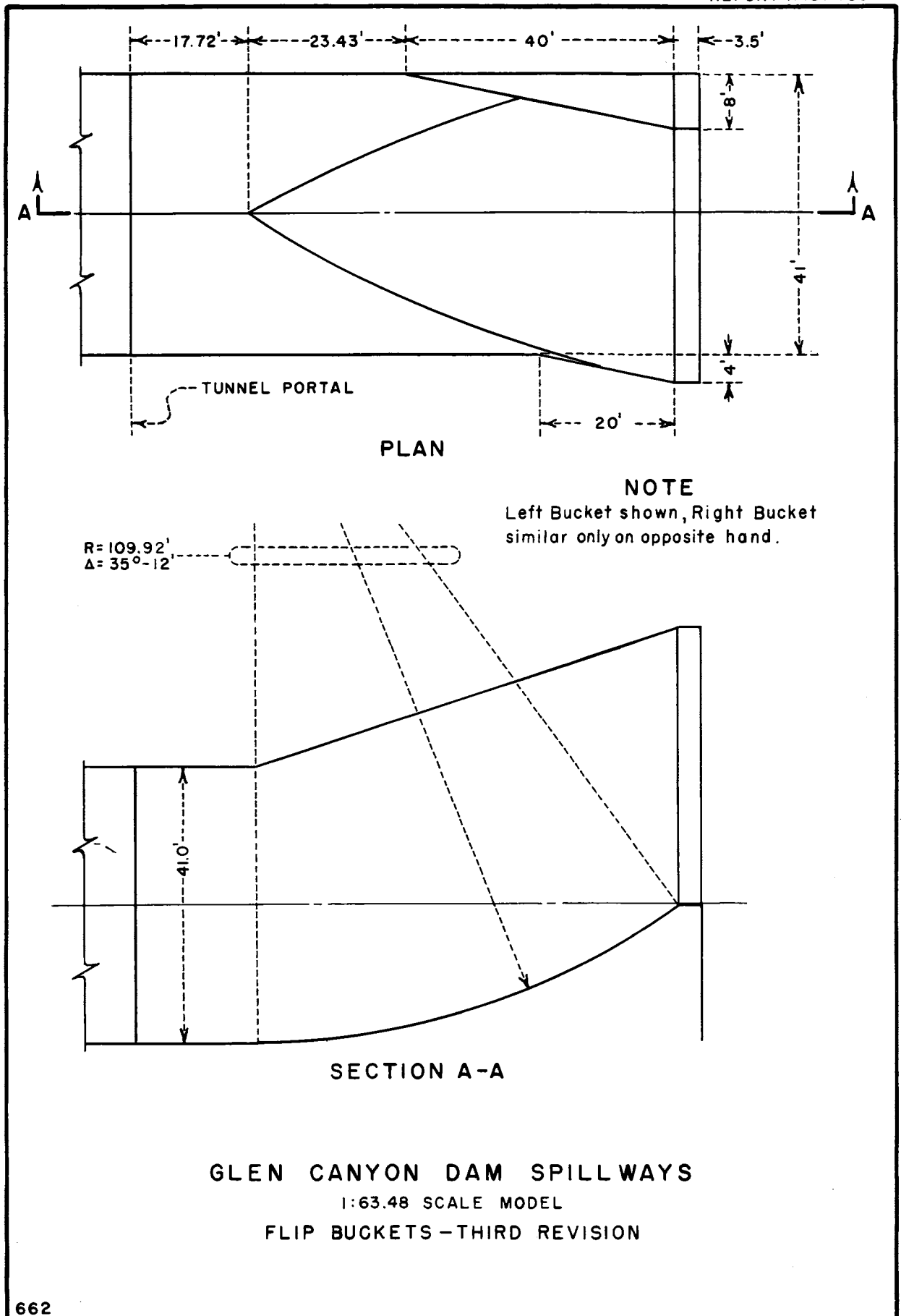
Figure 52
Report Hyd-469

FIGURE 53
REPORT HYD. 469

PLAN

NOTE
Left Bucket shown, Right Bucket
similar only on opposite hand.

R= 109.92'
Δ= 35°-12'

SECTION A-A

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
FLIP BUCKETS—THIRD REVISION

Discharge = 25,000 cfs

Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Left Flip Bucket - Third Revision

Figure 54
Report Hyd-469

Discharge = 25,000 cfs   Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Right Flip Bucket - Third Revision

Figure 55
Report Hyd-469

Figure 56
Report Hyd-469

Discharge = 25,000 cfs

Discharge = 75,000 cfs

Discharge = 138,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow from Both Flip Buckets - Third Revision

PLAN

PLAN

SECTION A-A
LEFT HAND BUCKET

SECTION B-B
RIGHT HAND BUCKET

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
FLIP BUCKETS—FOURTH REVISION

FIGURE 42
REPORT HYD. 469

4-5-63 D Q.T.L.	ADDED TUNNEL INVERT REPAIRS AND ELBOW TRANSITION NOTES
1-31-64 D-E.Q.L.	DELETED ELBOW TRANSITION NOTE.

Intersection of axes of crest-Sta. 20+00 (Along ℄ of tunnel)
El. 3648.000
El. 3638.122
Sta. 20+45.000
$X^2=-205Y$
P.T.-El. 3543.470
Sta. 21+46.385
El. 3506.677
Sta. 21+72.148
35°00'
48.250' Dia.
Tunnel dia. varies uniformly from 48'-3" at Sta. 21+72.148 to 41'-2 13/16" at Sta. 22+75.442
Min. concrete thickness varies uniformly from 3'-3" at Sta. 21+72.148 to 2'-9" at Sta. 22+82.323
El. 3359.157
Sta. 22+75.442
Sta. 22+82.323
El. 3349.331
P.C.-El. 3286.611
Sta. 23+26.240
308.117' R.
11.995'
Tunnel dia. varies from 41'-2 13/16" at Sta. 22+75.442 to 41'-0" at Sta. 22+82.323
41.0' Dia.
350' R.
Δ=2°13'52.23"
R=308.117'
T=6.000'
L=11.998'
Sta. 25+68.303
El. 3549.576
Sta. 26+12.943
El. 3487.363
Δ=54°47'58.08"
R=350'
T=181.421'
L=334.751'
Contraction joint plane
El. 3182.00
20'-6" R.
C
C
"A" Line
5.0'
3.0'
3.0'
"A" Line
"A" Line
Backfill concrete. For details, see Dwg. 557-D-2218 (557-D-2805, Left tunnel)
P.I.-El. 3138.000
Sta. 24+30.298
Sta. 26+11.718
P.C.-El. 3137.365
Min. concrete thickness-2'-9"
℄ Tunnel
"A" Line
Slope 0.0035
Right tunnel invert repairs not shown, see Dwg. 557-D-2807
For details of portal, see Dwg. 557-D-1982 (557-D-2099, Left tunnel)
El. 3182.00
B
B
Contraction joint
Downstream portal
*El. 3134.179
El. 3133.570 (Left tunnel)
*Sta. 35+22.00
Sta. 36+96.00 (Left tunnel)
Y
X
Origin of equations
El. 3648.000
Intersection of axes of crest-Sta. 20+00 (Along ℄ of tunnel)
El. 3638.122
Sta. 20+45.000
Keyways not shown
Base line: $X^2=-205Y$
$Y=-0.00000150621X^4+0.000062176X^3-0.01180053X^2+0.42883X-10.3275$
X=59.874
Y=24.001
For details of intake structure, see Dwg. 557-D-602
0.4390
Roof line: $Y=-0.000000185202X^4+0.0000901957X^3-0.0199521X^2+1.296315X+16.637429$
Sta. 21+25.500
Spring line: $Y=-0.00394196X^2-0.173513X+48.52100$
El. 3645.417
P.T. of pier nose
El. 3626.482
A
A
A
A
P.C. of side wall curve
El. 3607.222
Sta. 20+91.430
El. 3599.220
Sta. 21+00.000
El. 3590.458
Sta. 21+08.610
"A" Line
"A" Line
P.T. X=187.343 Y=-75.851
P.T. X=166.864 Y=-90.190
P.T.-El. 3543.470
Sta. 21+46.385
X=191.910 Y=-127.486 P.T.
48.25' Dia.
35°00'
El. 3506.677
Sta. 21+72.148
Lining thickness to "A" line varies uniformly from 8'-6" at Sta. 20+45.000 to 5'-10" at Sta. 20+92.770; from 5'-10" at Sta. 20+92.770 to 4'-3" at Sta. 21+30.000; from 4'-3" at Sta. 21+30.000 to 3'-3" at Sta. 21+72.148.
"A" Line
$R_1=\frac{W^2+4h^2}{8h}$
h
H
T
W
R_1
R_2
R_2
P.T. of pier nose
Radius varies
℄ Pier

SECTION B-B

SECTION C-C

ELEVATION - RIGHT SPILLWAY TUNNEL
(LEFT SPILLWAY TUNNEL IDENTICAL EXCEPT AS NOTED)

ELEVATION-TRANSITION
STA. 20+45.000 TO STA. 21+72.148

TYPICAL TRANSITION SECTION
STA. 20+45.000 TO STA. 21+72.148
SECTION NORMAL TO BASE LINE

SECTION A-A

Axes of crest
Intersection of axes of crest with ℄ of tunnel-Sta. 20+00
Sta. 20+45.000
8.500'
46.406'
114.135'
231.679'
88.941'
70.150'
Tunnel portal
"A" Line
Transition
P.C. of side wall curve-Sta. 21+00.000
$Y=-0.00000203456X^3-0.00086594X^2+0.01948X-24.125$
Origin of equation Sta. 21+72.148
℄ Tunnel
Values of "Y" are symmetrical about ℄
(-Y)
(-X)
"A" Line
W=2Y
48.250' Dia.
$W=70.149894+0.277443(114.134777-S)$
S=Developed length of base line from Sta. 20+00 along ℄ of tunnel

SECTIONAL PLAN-TRANSITION
STA. 20+45.000 TO STA. 21+72.148
(DEVELOPED ALONG BASE LINE)

BASE LINE		ROOF LINE		ELEMENTS OF TRANSITION SECTIONS					
Station	Elevation	Station	Elevation	H	h	W	R_1	R_2	T
20+45.000	3638.122	20+65.903	3685.735	52'-0"	15'-0"	88'-11 9/16"	73'-5 1/16"	0	8'-6"
20+48.650	3636.455	20+71.051	3683.651	52'-2 15/16"	16'-0 9/16"	87'-9 13/16"	68'-1 5/16"	5 3/16"	8'-3 7/8"
20+52.230	3634.693	20+75.995	3681.331	52'-4 1/8"	16'-10 7/8"	86'-8 5/8"	64'-0 11/16"	10 7/16"	8'-1 11/16"
20+55.750	3632.839	20+80.763	3678.826	52'-4 3/16"	17'-7 9/16"	85'-7 3/8"	60'-9 1/2"	15 7/16"	7'-11 9/16"
20+59.250	3630.875	20+85.418	3676.144	52'-3 7/16"	18'-3"	84'-6 1/16"	58'-0 7/16"	20 1/2"	7'-9 7/16"
20+62.680	3628.835	20+89.905	3673.356	52'-2 1/4"	18'-9 7/16"	83'-4 3/4"	55'-8"	2'-1 9/16"	7'-7 5/16"
20+66.080	3626.700	20+94.288	3670.454	52'-0 11/16"	19'-3 1/4"	82'-3 3/8"	53'-6 5/8"	2'-6 7/8"	7'-5 1/8"
20+69.440	3624.478	20+98.561	3667.464	51'-11 1/16"	19'-8 7/16"	81'-2"	51'-7 11/16"	3'-0 3/8"	7'-3"
20+72.710	3622.211	21+02.671	3664.448	51'-9 7/16"	20'-1 1/4"	80'-0 3/4"	49'-10 7/8"	3'-6"	7'-0 7/8"
20+75.940	3619.869	21+06.689	3661.372	51'-7 13/16"	20'-5 11/16"	78'-11 7/16"	48'-3 9/16"	3'-11 15/16"	6'-10 11/16"
20+79.140	3617.448	21+10.632	3658.236	51'-6 3/8"	20'-9 13/16"	77'-10 1/16"	46'-9 9/16"	4'-6 3/16"	6'-8 9/16"
20+82.300	3614.960	21+14.494	3655.056	51'-5 1/16"	21'-2"	76'-8 11/16"	45'-4 1/8"	5'-0 11/16"	6'-6 7/16"
20+85.370	3612.449	21+18.219	3651.889	51'-3 13/16"	21'-5 7/8"	75'-7 1/2"	44'-0 1/8"	5'-7 7/16"	6'-4 5/16"
20+88.430	3609.854	21+21.906	3648.657	51'-3"	21'-9 11/16"	74'-6 1/8"	42'-8 3/4"	6'-2 9/16"	6'-2 1/8"
20+91.430	3607.222	21+25.500	3645.417	51'-2 3/16"	22'-1 5/16"	73'-4 13/16"	41'-6 1/4"	6'-9 13/16"	6'-0"
20+94.400	3604.530	21+29.037	3642.139	51'-1 9/16"	22'-4 13/16"	72'-3 1/2"	40'-4 5/16"	7'-5 3/8"	5'-9 7/8"
20+97.310	3601.809	21+32.485	3638.860	51'-1 1/16"	22'-8 1/4"	71'-2 1/4"	39'-3 3/16"	8'-1 9/16"	5'-7 11/16"
21+00.000	3599.220	21+35.656	3635.767	51'-0 11/16"	22'-11 9/16"	70'-1 13/16"	38'-3 3/8"	8'-9 1/8"	5'-5 11/16"
21+00.200	3599.024	21+35.891	3635.535	51'-0 11/16"	22'-11 9/16"	70'-0 7/8"	38'-2 1/2"	8'-9 3/4"	5'-5 9/16"
21+03.040	3596.209	21+39.222	3632.201	51'-0 7/16"	23'-2 3/4"	68'-11 5/8"	37'-2 9/16"	9'-6 1/8"	5'-3 7/16"
21+05.860	3593.335	21+42.512	3628.824	51'-0 1/4"	23'-5 3/4"	67'-10 7/16"	36'-3 1/8"	10'-2 3/4"	5'-1 1/4"
21+08.610	3590.458	21+45.705	3625.466	51'-0 1/16"	23'-8 5/8"	66'-9 1/2"	35'-4 7/16"	10'-11 1/2"	4'-11 1/8"
21+11.310	3587.561	21+48.823	3622.105	50'-11 15/16"	23'-11 5/8"	65'-8 7/8"	34'-6 7/16"	11'-8 5/16"	
21+14.000	3584.605	21+51.912	3618.692	50'-11 13/16"	24'-1 13/16"	64'-8 1/4"	33'-8 13/16"	12'-5 1/4"	
21+16.670	3581.601	21+54.961	3615.241	50'-11 5/8"	24'-4 1/8"	63'-7 13/16"	32'-11 11/16"	13'-2 3/8"	

BASE LINE		ROOF LINE		ELEMENTS OF TRANSITION SECTIONS				
Station	Elevation	Station	Elevation	H	h	W	R_1	R_2
21+19.300	3578.573	21+57.946	3611.777	50'-11 3/8"	24'-6 1/4"	62'-7 5/8"	32'-3 1/8"	13'-11 7/16"
21+21.900	3575.514	21+60.878	3608.288	50'-11 1/8"	24'-8 1/8"	61'-7 11/16"	31'-7"	14'-8 5/16"
21+24.470	3572.425	21+63.756	3604.777	50'-10 11/16"	24'-9 3/4"	60'-7 13/16"	30'-11 3/8"	15'-5 5/8"
21+26.970	3569.359	21+66.536	3601.300	50'-10 3/16"	24'-11 1/8"	59'-8 3/4"	30'-4 1/4"	16'-2 7/16"
21+29.480	3566.219	21+69.307	3597.747	50'-9 9/16"	25'-0 1/4"	58'-9 5/8"	29'-9 5/8"	16'-11 1/4"
21+31.960	3563.056	21+72.023	3594.175	50'-8 3/4"	25'-1 1/16"	57'-10 13/16"	29'-3"	17'-7 13/16"
21+34.380	3559.912	21+74.651	3590.630	50'-7 13/16"	25'-1 9/16"	57'-0 1/2"	28'-9"	18'-4 1/4"
21+36.800	3556.711	21+77.258	3587.025	50'-6 5/8"	25'-1 7/8"	56'-2 7/16"	28'-3 1/4"	19'-0 1/2"
21+39.180	3553.507	21+79.798	3583.420	50'-5 5/16"	25'-1 7/8"	55'-4 3/4"	27'-9 13/16"	19'-8 3/8"
21+41.530	3550.289	21+82.283	3579.804	50'-3 15/16"	25'-1 9/16"	54'-7 9/16"	27'-4 7/8"	20'-4"
21+43.880	3547.017	21+84.745	3576.130	50'-2 1/8"	25'-1"	53'-10 5/8"	27'-0 1/8"	20'-11 3/8"
21+46.180	3543.763	21+87.131	3572.477	50'-0 3/16"	25'-0 1/16"	53'-2 1/4"	26'-7 3/4"	21'-6 5/16"
21+46.385	3543.470	21+87.343	3572.149	50'-0"	25'-0"	53'-1 1/2"	26'-7 5/16"	21'-6 15/16"
21+48.470	3540.493	21+89.312	3569.090	49'-10 5/16"	24'-11 1/8"	52'-6 9/16"	26'-3 9/16"	22'-0 3/16"
21+50.770	3537.208	21+91.484	3565.716	49'-8 7/16"	24'-10 3/16"	51'-10 3/4"	25'-11 11/16"	22'-6 1/8"
21+53.060	3533.938	21+93.646	3562.356	49'-6 9/16"	24'-9 1/4"	51'-3 3/4"	25'-8 1/16"	22'-10 15/16"
21+55.360	3530.653	21+95.818	3558.982	49'-4 11/16"	24'-8 5/16"	50'-9 1/4"	25'-4 3/4"	23'-3 1/8"
21+57.650	3527.382	21+97.981	3555.622	49'-2 13/16"	24'-7 7/16"	50'-3 5/16"	25'-1 3/4"	23'-6 5/8"
21+59.950	3524.098	22+00.153	3552.248	49'-0 15/16"	24'-6 1/2"	49'-9 7/8"	24'-11"	23'-9 1/2"
21+62.240	3520.827	22+02.315	3548.888	48'-11 1/16"	24'-5 9/16"	49'-5"	24'-8 1/2"	23'-11 5/8"
21+64.530	3517.557	22+04.478	3545.529	48'-9 3/16"	24'-4 5/8"	49'-0 3/4"	24'-6 5/8"	24'-1 1/8"
21+66.830	3514.272	22+06.650	3542.154	48'-7 5/16"	24'-3 11/16"	48'-9 1/16"	24'-4 9/16"	24'-2"
21+69.120	3511.001	22+08.813	3538.795	48'-5 7/16"	24'-2 3/4"	48'-6 1/16"	24'-3"	24'-2 3/16"
21+71.420	3507.717	22+10.985	3535.420	48'-3 9/16"	24'-1 13/16"	48'-3 3/8"	24'-1 13/16"	24'-1 3/4"
21+72.148	3506.677	22+11.672	3534.352	48'-3"	24'-1 1/2"	48'-3"	24'-1 1/2"	24'-1 1/2"

TRANSITION DATA
STA. 20+45.000 TO STA. 21+72.148
SECTIONS NORMAL TO BASE LINE

2-17-59 D E.Q.L.	RELOCATED LEFT TUNNEL PORTAL FROM STA. 36+94 TO STA. 36+96 AND REVISED INVERT ELEVATION
12-29-58 D E.Q.L.	RELOCATED LEFT TUNNEL PORTAL FROM STA. 36+54 TO STA. 36+94 AND REVISED INVERT ELEVATION.

CONCRETE REQUIREMENTS

FINISHES
Tunnel below ℄ or spring line .... F4 or U3
Tunnel below ℄ from Sta. 23+20
to Sta. 26+15 .... F4 with special finish or U3
All other surfaces .... F2 or U2

STRENGTH
Concrete design is based on a compressive strength of not less than 3000 lbs. per sq. inch at 28 days.

NOTES

Reinforcement required but not shown
Chamfer all exposed corners 3/4" unless otherwise specified.
"A" Line is line of minimum concrete thickness.

1-13-56 D E.Q.L.	TRACED

THIS DRAWING SUPERSEDES DWGS. 557-D-86 AND 87 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.- GLEN CANYON UNIT—ARIZONA-UTAH

GLEN CANYON DAM
SPILLWAYS
RIGHT AND LEFT TUNNELS
ELEVATIONS AND SECTIONS

DRAWN J.L. SUBMITTED E. R. Schultz
TRACED V.M.M. RECOMMENDED O. L. Rice
CHECKED A.T.L. APPROVED K. B. Keener
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, DEC. 5, 1957
557-D-444

Left gate full open, Q = 5,760

Left gate 75% open, Q = 3,630

Left gate 50% open, Q = 2,290

A. 100 Foot Head

Left gate 25% open, Q = 1,150

Left gate full open, Q = 10,770

Left gate 75% open, Q = 6,780

Left gate 50% open, Q = 4,290

B. 350 Foot Head

Left gate 25% open, Q - 2,150

GLEN CANYON
LEFT DIVERSION TUNNEL OUTLET WORKS

Tunnel Flow Conditions With Parallel Conduits - Left Gate Only
1:24 Model

FIGURE 58
REPORT HYD. 469

PLAN

SCALE OF FEET

ELEVATION ALONG ℄ 41' DIA. CONCRETE LINED DIVERSION TUNNEL

SECTIONAL ELEVATION THRU OUTLET GATES

SCALE OF FEET

Note: Reinforcement for gate operating chamber, 7'x5' access adit, gates and conduits not shown. Gates to be left in place and gate operating chamber, downstream section of conduits, 7'x5' access adit, and 5'x7' access adit to dam to be backfilled and grouted before final section of tunnel plug is placed.

SECTION A-A

SCALE OF FEET

SECTION C-C

SCALE OF FEET

SECTION B-B

12-14-62	D.E.C.L.	REVISED TOP OF DAM AND SPILLWAY DEFLECTOR BUCKET
9-21-59	D.E.C.L.	WIDENED ACCESS ADIT IN TUNNEL PLUG AND REVISED NOTE
1-23-59	D.E.C.L.	REVISED TO CONFORM TO THE DETAIL DRAWINGS TO DATE.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT--ARIZONA-UTAH
GLEN CANYON DAM
LEFT DIVERSION TUNNEL
OUTLET WORKS

DRAWN _____ SUBMITTED _____
TRACED _J.F.M._ RECOMMENDED _____
CHECKED _____ APPROVED _____

DENVER, COLORADO, JAN. 30, 1958   557-D-600

PLAN
STA. 36 + 96
PORTAL

A B C D E F
15
16-19 22-25 26-29 30-32
20-21
1 2 3 4 5 6
7
10 12
8 9 11 13 14

SECTION G-G
El. 3184.48
El. 3171.26
El. 3160.68
El. 3155.39
El. 3157.0
P.C. El. 3133.57

19 25 29 32
18 24 28 31
17 21 23 27 30
16 20 22 26

SECTION C-C
STA. 37+40
El. 3189.30
El. 3160.0
El. 3160.68 — 21
El. 3155.39 — 20
El. 3149.20
El. 3142.78

SECTION F-F
STA. 37+63.5
El. 3197.16
El. 3184.48 — 32
El. 3171.25 — 31
El. 3160.0
El. 3160.68 — 30
El. 3157.0

Note: For Details of Bucket
See Figures 60,61,62,63

SECTION D-D
STA. 37+45
El. 3190.97
El. 3184.48 — 25
El. 3171.26 — 24
El. 3160.68 — 23
El. 3155.39 — 22
El. 3149.2

SECTION A-A
STA. 37+26
El. 3160.0
El. 3155.39
El. 3143.92
El. 3137.72

SECTION B-B
STA. 37+35
El. 3187.62
El. 3184.48 — 19
El. 3171.26 — 18
El. 3160.68 — 17
El. 3155.39 — 16
El. 3160.0
El. 3140.72

SECTION E-E
STA. 37+54
El. 3194.0
El. 3184.48 — 29
El. 3171.28 — 28
El. 3160.68 — 27
El. 3155.39 — 26
El. 3160.0
El. 3150.24

PRESSURES IN FEET OF WATER

PIEZ	Q = 138,000	Q = 100,000	Q = 75,000	Q = 50,000
1	211.3	166.6	132.9	95.2
2	205.0	164.7	132.3	93.9
3	206.3	164.0	133.6	95.8
4	146.8	113.7	91.3	64.1
5	34.4	27.8	25.1	17.9
6	0	0	0	0
7	119.7	27.4	44.0	19.8
8	53.6	11.5	1.0	—
9	67.4	29.8	6.6	0.4
10	132.3	106.8	59.5	29.8
11	65.5	31.7	7.3	—
12	150.8	119.0	84.6	52.9
13	82.3	50.9	27.8	9.9
14	99.9	80.7	60.2	39.7
15	138.9	80.7	6.0	—
16	133.9	74.0	25.1	—
17	71.4	14.5	-1.4	—
18	-4.6	-0.7	—	—
19	—	—	—	—
20	143.5	88.6	46.3	9.9
21	91.3	37.7	11.2	4.0
22	138.9	90.6	56.9	22.5
23	92.6	47.6	25.8	6.0
24	17.2	4.0	3.3	0.7
25	-1.6	—	—	—
26	123.0	94.8	76.4	52.9
27	90.3	60.8	43.6	25.1
28	25.1	5.3	1.3	1.0
29	-7.6	-4.6	-3.3	-1.6
30	30.4	22.8	22.5	17.2
31	21.8	10.6	5.3	3.0
32	-3.3	-4.0	-3.3	-2.6
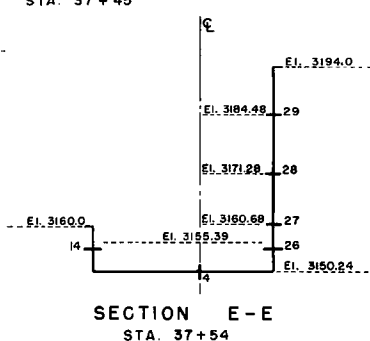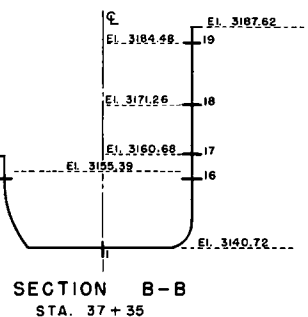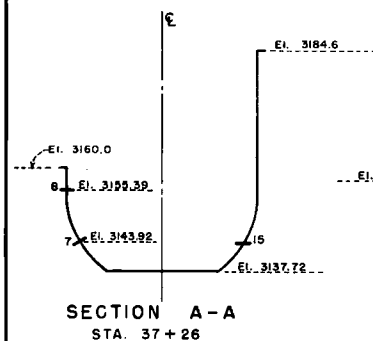
GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PIEZOMETERS AND PRESSURES IN RECOMMENDED LEFT BUCKET
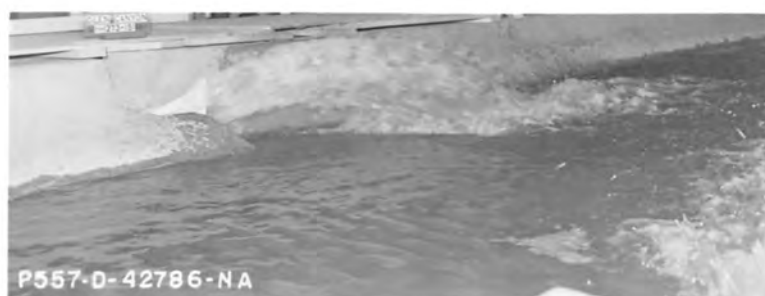
Figure 65
Report Hyd-469

Discharge = 25,000 cfs
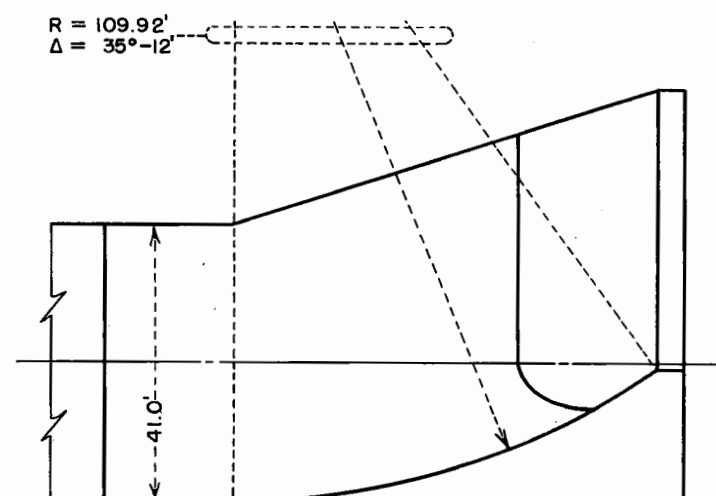

Discharge = 75,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Recommended Left Flip Bucket

Figure 66
Report Hyd-469

Discharge = 25,000 cfs

Discharge = 75,000 cfs

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Flow in Recommended Right Flip Bucket

Discharge = 138,000 cfs each bucket
Tailwater Elevation = 3180.0

Left Bucket

Right Bucket

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Operation of Recommended Flip Buckets
at Maximum Discharge

Figure 67
Report Hyd-469

WATER SURFACE

ELEVATION

3350
3300
3200
3100

35 + 00   37 + 00   39 + 00   41 + 00   43 + 00   45 + 00

STATIONS IN FEET
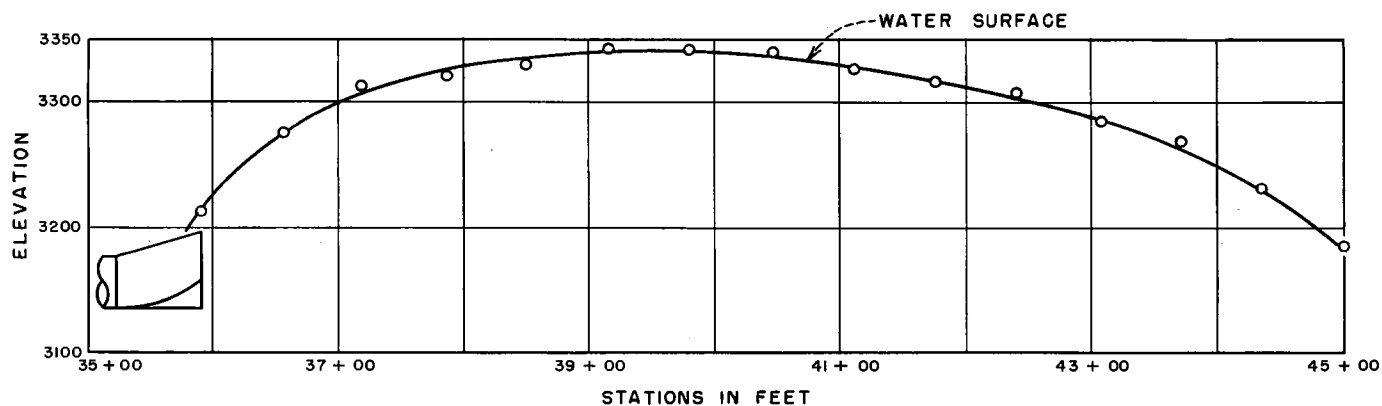
RIGHT SPILLWAY

Note: Profiles taken at maximum
discharge, 138,000 c.f.s. each
spillway, T.W. elev. 3180.0.

WATER SURFACE

ELEVATION

3350
3300
3200
3100

35 + 00   37 + 00   39 + 00   41 + 00   43 + 00   45 + 00

STATIONS IN FEET
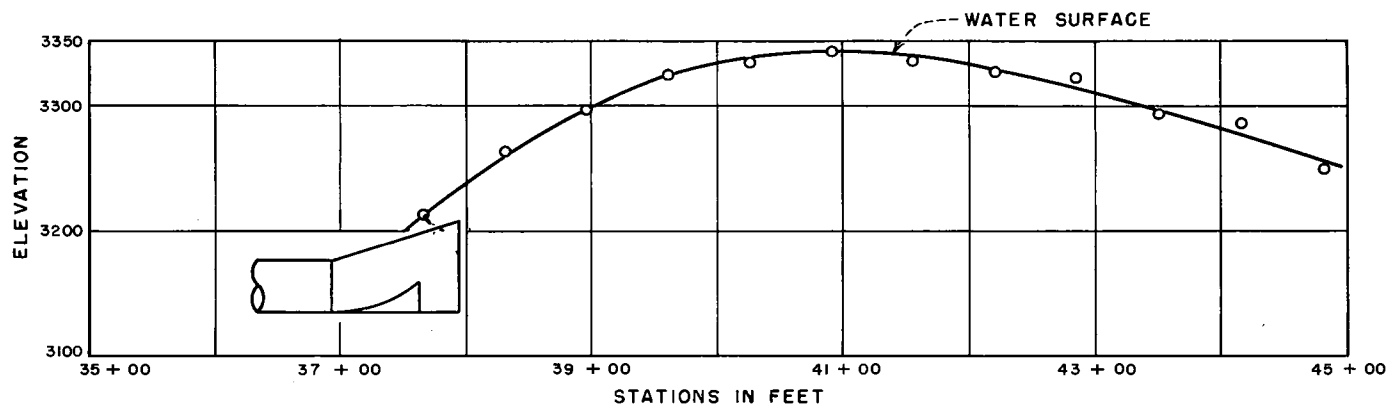
LEFT SPILLWAY

GLEN CANYON DAM SPILLWAYS

1:63.48 SCALE MODEL

WATER SURFACE PROFILES FROM RECOMMENDED FLIP BUCKETS

FIGURE 68
REPORT HYD. 469

FIGURE 60
REPORT HYD. 469

PLAN

SECTION J-J

SECTION B-B

SECTION A-A

SECTION C-C

CONCRETE REQUIREMENTS
FINISHES:
Interior surface along and above final invert ____ F4 or U3
Construction joint surfaces ____ F1 or U1
All other surfaces ____ F2 or U2

STRENGTH:
Design of concrete is based on a compressive strength of 3000 lbs. per sq. inch at 28 days.

NOTES
Chamfer ½" or tool all exposed corners unless otherwise specified.
Reinforcement required but not shown.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON DAM
RIGHT SPILLWAY-DOWNSTREAM PORTAL
DEFLECTOR BUCKET
PLAN AND SECTIONS

DENVER, COLORADO   JUNE 25, 1962
557-D-1982

FIGURE 61
REPORT HYD. 469

DETAIL Z (1982)

Grating not shown.

8"C.I. pipe drain

SECTION H-H

SECTION D-D (1982)
TYPICAL FROM STA. 35+22 TO STA. 35+52

SECTION E-E (1982)
TYPICAL FROM STA. 35+52 TO STA. 35+67.96

SECTION F-F (1982)
TYPICAL FROM STA. 35+67.96 TO STA. 35+92

SECTION G-G (1982)

TRANSITION DATA

STATION	El. A	El. B	E
35+22	3134.43	3176.18	0
35+29.38	3134.43	3177.65	0
35+31.0	3134.55	3178.19	2'-3⅜"
35+33.0	3134.74	3178.86	3'-7⅞"
35+35.0	3134.96	3179.53	4'-9⅜"
35+37.0	3135.22	3180.20	5'-9⅜"
35+39.34	3135.57	3180.98	6'-11⅞"
35+41.25	3135.89	3181.62	7'-10⅜"
35+43.19	3136.26	3182.27	9'-0"
35+43.50	3136.32	3182.38	9'-1⅜"
35+46	3136.86	3183.21	10'-1⅜"
35+49	3137.58	3184.22	11'-3⅜"
35+52	3138.39	3185.22	12'-5⅜"

STATION	El. A	El. B	El. C	E	F	G
35+54	3138.98	3185.89	3150.91	13'-2⅜"	20'-1⅜"	20'-6⅜"
35+56	3139.62	3186.56	3149.37	13'-10⅜"	19'-9⅜"	20'-6⅜"
35+58.50	3140.48	3187.40	3147.95	14'-9⅜"	19'-4⅜"	20'-7⅜"
35+61	3141.40	3188.23	3146.80	15'-7⅜"	18'-11⅜"	20'-8⅜"
35+63.50	3142.39	3189.07	3145.82	16'-4⅜"	18'-5⅜"	20'-9⅜"
35+66	3143.46	3189.91	3144.96	17'-1⅜"	18'-0⅜"	20'-11⅜"
35+67.96	3144.35	3190.56	3144.35	17'-8⅜"	17'-8⅜"	21'-1⅜"
35+68.50	3144.60	3190.74		17'-7⅜"	21'-2⅜"	
35+71	3145.82	3191.58		17'-2⅜"	21'-4⅜"	
35+73.50	3147.12	3192.42		16'-8⅜"	21'-7⅜"	
35+76	3148.50	3193.25		16'-3⅜"	21'-11⅜"	
35+78.50	3149.97	3194.09		15'-10⅜"	22'-3"	
35+81	3151.54	3194.93		15'-5⅜"	22'-7⅜"	
35+83.50	3153.20	3195.76		14'-11⅜"	22'-11⅜"	
35+85.61	3154.68	3196.47		14'-7⅜"	23'-3⅜"	
35+86	3154.96	3196.60		14'-6⅜"	23'-4⅜"	
35+89.50	3157.61	3197.77		13'-4⅛"	24'-0⅜"	
35+92	3157.11	3198.61		13'-6"	24'-6"	

VALUES OF OFFSET H

STA.	El. D	35+54	35+56	35+58.50	35+61	35+63.50	35+66	35+67.96	35+68.50	35+71	35+73.50	35+76	35+78.50	35+81	35+83.50	35+85.61
35+54	3138.98	13'-2⅜"														
35+56	3139.62	13'-11"	13'-11⅜"													
35+58.50	3140.48	14'-9⅜"	14'-9⅜"	14'-10⅜"												
35+61	3141.40	15'-7⅜"	15'-7⅜"	15'-8⅜"	15'-9⅜"											
35+63.50	3142.39	16'-5"	16'-5⅜"	16'-6⅜"	16'-7⅜"	16'-8⅜"										
35+66	3143.46	17'-2"	17'-2⅜"	17'-3⅜"	17'-4⅜"	17'-5⅜"	17'-7⅜"									
35+67.96	3144.35	17'-8⅜"	17'-9"	17'-9⅜"	17'-10⅜"	18'-0⅜"	18'-2⅜"	18'-4"								
35+68.50	3144.60	17'-10⅜"	17'-10⅜"	17'-11⅜"	18'-0⅜"	18'-2⅜"	18'-4"	18'-5⅜"	18'-6⅜"							
35+71	3145.82	18'-6"	18'-6⅜"	18'-7⅜"	18'-8⅜"	18'-9⅜"	18'-11⅜"	19'-1⅜"	19'-1⅜"	19'-4⅜"						
35+73.50	3147.12	19'-0⅜"	19'-1⅜"	19'-3"	19'-4⅜"	19'-6⅜"	19'-8⅜"	19'-8⅜"	19'-11⅜"	20'-2⅜"						
35+76	3148.50	19'-6⅜"	19'-7"	19'-7⅜"	19'-9"	19'-10⅜"	20'-0⅜"	20'-2⅜"	20'-2⅜"	20'-5⅜"	20'-8⅜"	20'-11⅜"				
35+78.50	3149.97	19'-11⅜"	19'-11⅜"	20'-0⅜"	20'-1⅜"	20'-3⅜"	20'-5⅜"	20'-7"	20'-7⅜"	20'-10⅜"	21'-1⅜"	21'-4⅜"	21'-8⅜"			
35+81	3151.54	20'-3⅜"	20'-3⅜"	20'-4⅜"	20'-5⅜"	20'-7⅜"	20'-10⅜"	21'-1⅜"	21'-1⅜"	21'-4⅜"	21'-7⅜"	21'-10⅜"	22'-0"	22'-4⅜"		
35+83.50	3153.20	20'-5⅜"	20'-5⅜"	20'-6⅜"	20'-7⅜"	20'-9⅜"	20'-11⅜"	21'-0⅜"	21'-1⅜"	21'-4⅜"	21'-7⅜"	21'-10⅜"	22'-2⅜"	22'-6⅜"	22'-11"	
35+85.61	3154.68	20'-6⅜"	20'-6⅜"	20'-7⅜"	20'-8⅜"	20'-9⅜"	20'-11⅜"	21'-1⅜"	21'-2⅜"	21'-4⅜"	21'-7⅜"	21'-11⅜"	22'-3"	22'-7⅜"	22'-11⅜"	23'-3⅜"

NOTE
For general notes and concrete requirements,
see Dwg. 557-D-1982.

4-5-63	REVISED INVERT TO MATCH SPILLWAY TUNNEL REPAIRS

THIS DRAWING SUPERSEDES DWG. 557-D-86 IN PART

FIGURE 62
REPORT HYD. 469

PLAN

SECTION A-A

SECTION H-H

SECTION B-B

SECTION C-C

CONCRETE REQUIREMENTS

FINISHES:
All interior surfaces except as noted_____F4 or U3
Construction joint surfaces_____F1 or U1
All other surfaces_____F2 or U2

STRENGTH:
Design of concrete is based on a compressive
strength of 9000 lbs. per sq. inch at 28 days.

NOTES
Chamfer ¾" or tool all exposed corners unless
otherwise specified.
Reinforcement required but not shown.

THIS DRAWING SUPERSEDES DWG. 557-D-87 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.- GLEN CANYON UNIT- ARIZONA-UTAH

GLEN CANYON DAM
LEFT SPILLWAY - DOWNSTREAM PORTAL
DEFLECTOR BUCKET
PLAN AND SECTIONS

DRAWN.R.E.Q.   SUBMITTED.
TRACED.   RECOMMENDED.
CHECKED.   APPROVED.

DENVER, COLORADO,   JUNE 30, 1961
SHEET 1 of 2

557-D-2099

FIGURE 63
REPORT HYD. 469

DETAIL Z (2099)

Grating not shown

8" C.I. pipe drain

SECTION J-J

Slope
El. 3175.57
El. 3174.57
El. 3173.50
8" C.I. pipe drain

SECTION D-D (2099)
TYPICAL FROM STA. 36+96 TO STA. 37+26

El. B
12"
23'-6"
8" C.I. pipe drain
₵ Spillway tunnel
5'-0"
El. 3160.00
El. 3159.50
El. 3154.07
20'-6" R.
Excavate as steep as practicable in sound rock
0.333:1
26.5 Min.
El. 3140.00
Line drill in sound rock below El. 3140.0
Excavation line
Line of minimum excavation
Remove existing concrete
Trace of structure Sta. 37+11 to Sta. 37+26
El. 3127.8, Sta. 36+96 to Sta. 37+11. Elev. varies uniformly from El. 3127.8 at Sta. 37+11 to El. 3130.0 at Sta. 37+26.

SECTION E-E (2099)
TYPICAL FROM STA. 37+26 TO STA. 37+41.96

El. B
12"
₵ Spillway tunnel
8" C.I. pipe drain
23'-6"
5'-0" (Radial)
El. 3160.00
El. 3159.50
El. 3154.07
F
G
H
El. D
20'-6" R.
El. C
El. A
E
0.333:1 (Radial)
El. 3140.00
Excavate as steep as practicable
Line drill
El. 3130.0
Line of minimum excavation

SECTION F-F (2099)
TYPICAL FROM STA. 37+41.96 TO STA. 37+66

El. B
12"
8" C.I. pipe drain
₵ Spillway tunnel
23'-6"
5'-0" (Radial)
El. 3160.00
El. 3159.50
F
G
El. 3154.07
H
El. A
El. D
0.333:1 (Radial)
El. 3140.00
El. 3130.0

SECTION G-G (2099)

El. 3208.04
12"
Excavation line
₵ Spillway tunnel
8" C.I. pipe drain
El. 3171.50
23'-6"
30" C.M.P.
El. 3167.00
9'-0"
24'-6"
8'-3"
El. 3159.50
El. 3160.00
El. 3156.50
El. 3157.00
6'-1⅛"
11'-9"
Elev. varies
El. 3140.00
96'-3"
El. 3130.0

TRANSITION DATA

STATION	El. A	El. B	E
36+96	3133.57	3175.57	0
36+98	3133.59	3175.57	10⅛
37+00	3133.64	3175.91	20¼
37+02.50	3133.76	3176.75	2'-9¼
37+05	3133.94	3177.58	5'-10⅛
37+07.50	3134.18	3178.42	4'-11⅛
37+10	3134.47	3179.26	6'-0¼
37+12.50	3134.83	3180.09	7'-0⅛
37+15	3135.24	3180.93	8'-1⅛
37+17.50	3135.71	3181.77	9'-1⅛
37+20	3136.25	3182.60	10'-1¼
37+23	3136.97	3183.61	11'-3⅛
37+26	3137.78	3184.61	12'-5⅛

STATION	El. A	El. B	El. C	E	F	G
37+28	3138.38	3185.28	3150.30	13'-2¼	20'-1⅛	20'-6⅛
37+30	3139.01	3185.95	3148.76	13'-10⅛	19'-9⅛	20'-6⅛
37+32.50	3139.87	3186.79	3147.34	14'-9⅛	19'-4⅛	20'-7⅛
37+35	3140.79	3187.62	3146.19	15'-7⅛	18'-11⅛	20'-8⅛
37+37.50	3141.78	3188.46	3145.21	16'-4⅛	18'-5⅛	20'-9⅛
37+40	3142.85	3189.30	3144.35	17'-1⅛	18'-0⅛	20'-11⅛
37+41.96	3143.74	3189.95	3143.74	17'-8⅛	17'-8⅛	21'-0⅛
37+42.50	3143.99	3190.15		17'-7⅛	21'-2⅛	
37+45	3145.21	3190.97		17'-2⅛	21'-4⅛	
37+47.50	3146.51	3191.81		16'-8⅛	21'-7⅛	
37+50	3147.89	3192.64		16'-3⅛	21'-11⅛	
37+52.50	3149.37	3193.48		15'-10⅛	22'-3⅛	
37+55	3150.93	3194.32		15'-5⅛	22'-7⅛	
37+57.50	3152.59	3195.15		14'-11⅛	22'-11⅛	
37+59.61	3154.07	3195.86		14'-7⅛	23'-3⅛	
37+60	3154.95	3195.99		14'-6⅛	23'-4⅛	
37+63.50	3157.00	3197.16		13'-11⅛	24'-0⅛	
37+66	3156.50	3198.00		13'-6"	24'-6⅛	
37+96		3208.04		8'-3"		

VALUES OF OFFSET H

STA. El. D	37+28	37+30	37+32.50	37+35	37+37.50	37+40	37+41.96	37+42.50	37+45	37+47.50	37+50	37+52.50	37+55	37+57.50	37+59.61
3138.38	15'-2⅛														
3139.01	13'-11⅛	13'-11⅛													
3139.87	14'-9⅛	14'-9⅛	14'-10⅛												
3140.79	15'-7⅛	15'-7⅛	15'-8⅛	15'-9⅛											
3141.78	16'-5	16'-5⅛	16'-6⅛	16'-7⅛	16'-8⅛										
3142.85	17'-2	17'-2⅛	17'-3⅛	17'-4⅛	17'-5⅛	17'-7⅛									
3143.74	17'-8⅛	17'-9	17'-9⅛	17'-10⅛	18'-0⅛	18'-2⅛	18'-4"								
3143.99	17'-10⅛	17'-10⅛	17'-11⅛	18'-0⅛	18'-2⅛	18'-4"	18'-5⅛	18'-6⅛							
3145.21	18'-6"	18'-6⅛	18'-7⅛	18'-8⅛	18'-9⅛	18'-11⅛	19'-1⅛	19'-1⅛	19'-4⅛						
3146.51	19'-0⅛	19'-1⅛	19'-1⅛	19'-3⅛	19'-4⅛	19'-6⅛	19'-8⅛	19'-8⅛	19'-11⅛	20'-2⅛					
3147.89	19'-6⅛	19'-7"	19'-7⅛	19'-9⅛	19'-10⅛	20'-0⅛	20'-2⅛	20'-2⅛	20'-5⅛	20'-8⅛	20'-11⅛				
3149.37	19'-11⅛	20'-0⅛	20'-1⅛	20'-3⅛	20'-5⅛	20'-7"	20'-7⅛	20'-10⅛	21'-1⅛	21'-4⅛	21'-8⅛				
3150.93	20'-3⅛	20'-4⅛	20'-5⅛	20'-7"	20'-8⅛	20'-10⅛	20'-11⅛	21'-1⅛	21'-4⅛	21'-7⅛	21'-10⅛	22'-0⅛	22'-4⅛		
3152.59	20'-5⅛	20'-5⅛	20'-6⅛	20'-7⅛	20'-9⅛	20'-11⅛	21'-0⅛	21'-1⅛	21'-4⅛	21'-7⅛	21'-10⅛	22'-2⅛	22'-6⅛	22'-11⅛	
3154.07	20'-6⅛	20'-6⅛	20'-7⅛	20'-8⅛	20'-9⅛	20'-11⅛	21'-1⅛	21'-2⅛	21'-4⅛	21'-7⅛	21'-11⅛	22'-3⅛	22'-7⅛	22'-11⅛	23'-3⅛

NOTE
For general notes and concrete requirements, see Dwg. 557-D-2099.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON DAM
LEFT SPILLWAY-DOWNSTREAM PORTAL
DEFLECTOR BUCKET
PLAN AND SECTIONS

DRAWN P.E.O. SUBMITTED E.B. Schutz
TRACED RECOMMENDED O.G. Rice
CHECKED J.K. A.F.K. APPROVED E.V. Lindseth
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JUNE 30, 1961
SHEET 2 OF 2
557-D-2100

FIGURE 69
REPORT HYD. 469

DETAIL Z

PROFILE-OUTLET NO.1

PROFILE-OUTLET NO. 2

PROFILE-OUTLET NO.3

PROFILE-OUTLET NO.4

NOTE
Galleries not shown

THIS DRAWING SUPERSEDES DWG. 557-D-96 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON DAM
RIVER OUTLETS
PROFILES

DENVER, COLORADO,   DEC. 26, 1957

557-D-566

Figure 70
Report Hyd-469

River outlets discharging 15,000 cfs.

Erosion after 3 hours (model time) operation
at 15,000 cfs.

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Operation of River Outlets, Preliminary Design

FIGURE 71
REPORT HYD. 469

96" HOLLOW JET VALVES

S 9° 00' W NO.4
S 14° 00' W NO.3
S 19° 00' W NO.2
S 24° 00' W NO.1

A. FIRST REVISION

96" HOLLOW JET VALVES

S 9° 00' W NO.4
S 16° 30' W NO.3
S 24° 00' W NO.2
S 31° 30' W NO.1
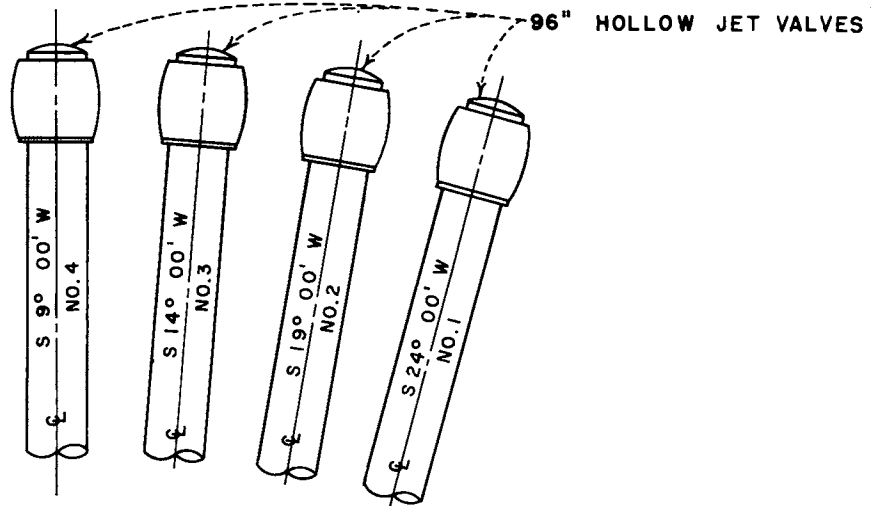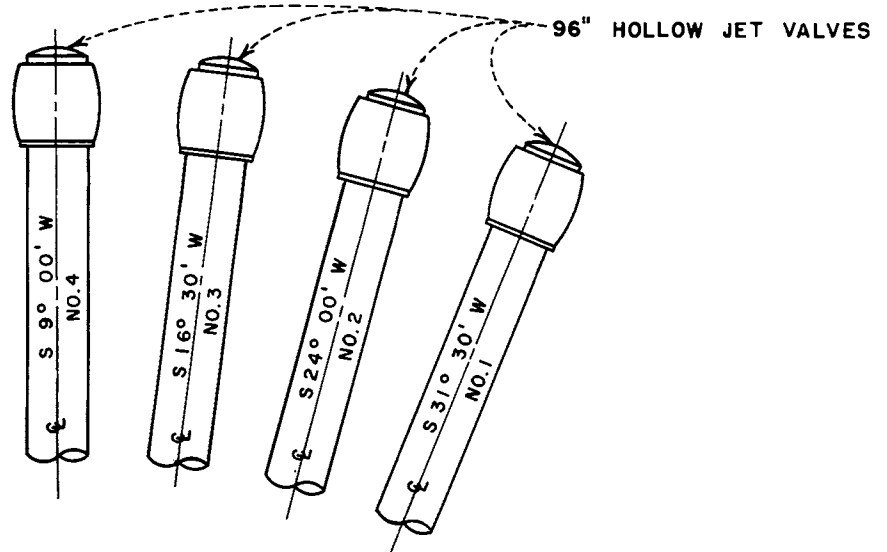
B. RECOMMENDED

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
RIVER OUTLETS — VALVE ALINEMENT

Figure 72
Report Hyd-469

Discharge = 15,000 cfs from River Outlets;
32,000 cfs from T.P.O.W., 24,000 cfs
through powerhouse.

Erosion after 10 hours operation under
above flow conditions.

GLEN CANYON DAM SPILLWAYS

1:63.48 Scale Model
Operation with Recommended Valve Alinement

FIGURE 73
REPORT HYD. 469
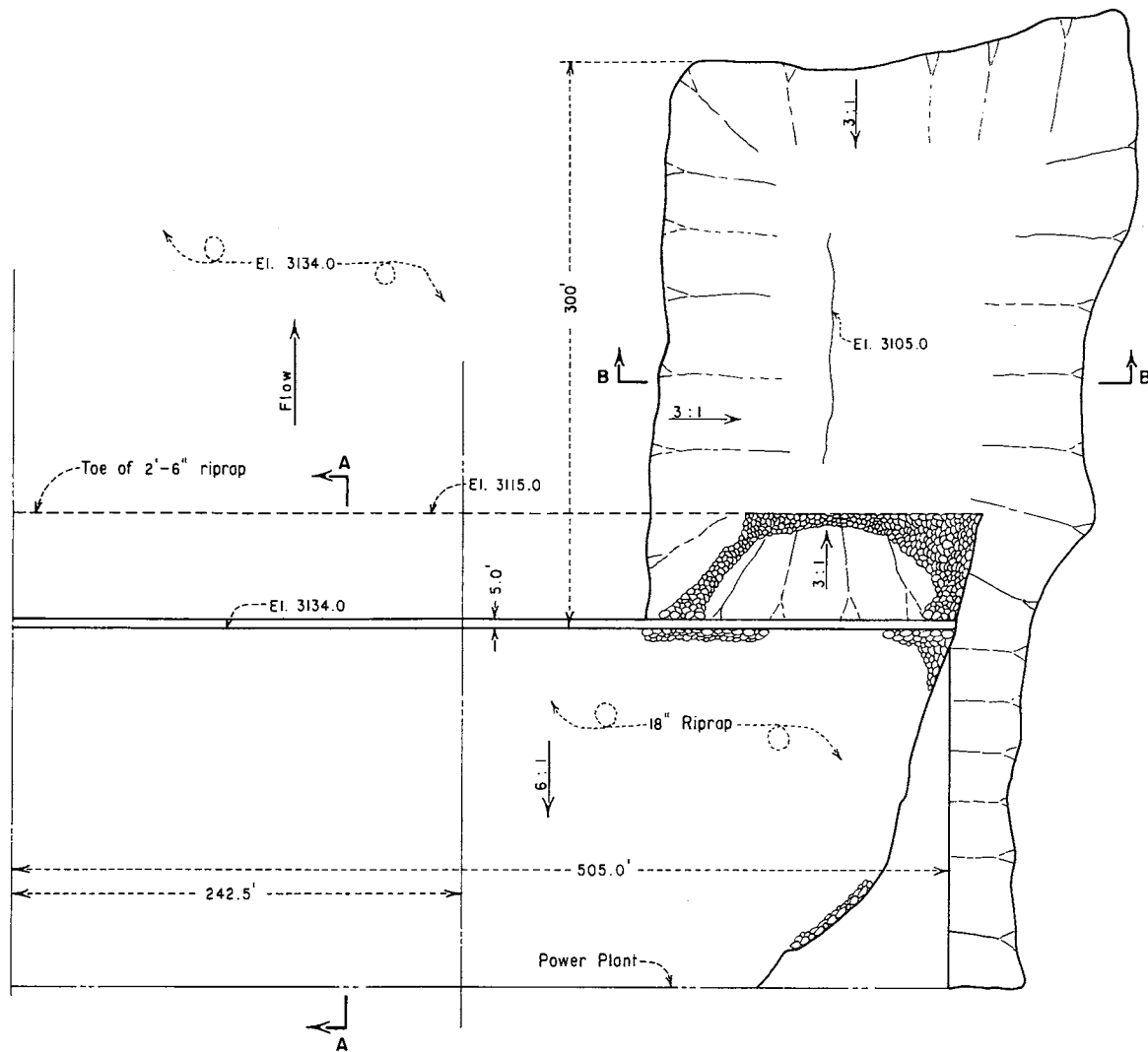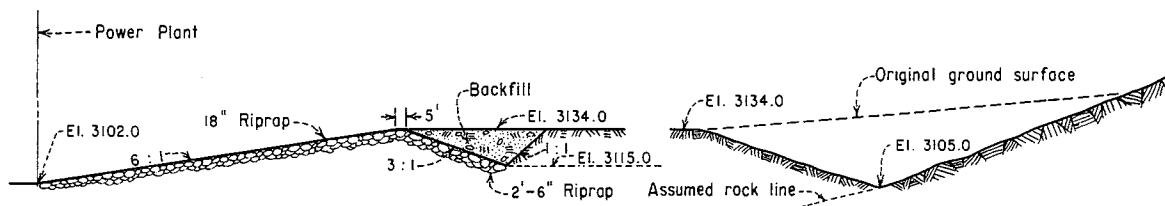
PLAN

SECTION A—A

SECTION B—B

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
PRE-EXCAVATED AREA IN AFTERBAY

Figure 74
Report Hyd-469

A.

Pre-excavated erosion hole along right bank in powerplant afterbay.
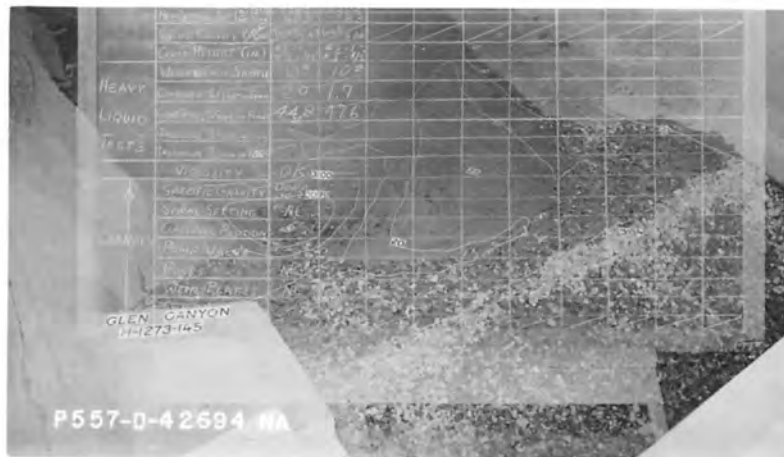
B.

Erosion after 16 hours operation, 24,000 cfs thru powerplant, 15,000 cfs thru river outlets, 32,000 cfs thru left spillway. No sand moved into afterbay.

C.

Erosion after 7 hours operation. Valves 3 and 4 discharging 7,500 cfs, 24,000 cfs thru powerplant. 32,000 cfs thru left spillway. About 5,000 yards of material moved into afterbay.

GLEN CANYON DAM SPILLWAYS

1:63.48 Model Studies
Results of Erosion Studies in Powerplant Afterbay

FIGURE 75
REPORT HYD. 469
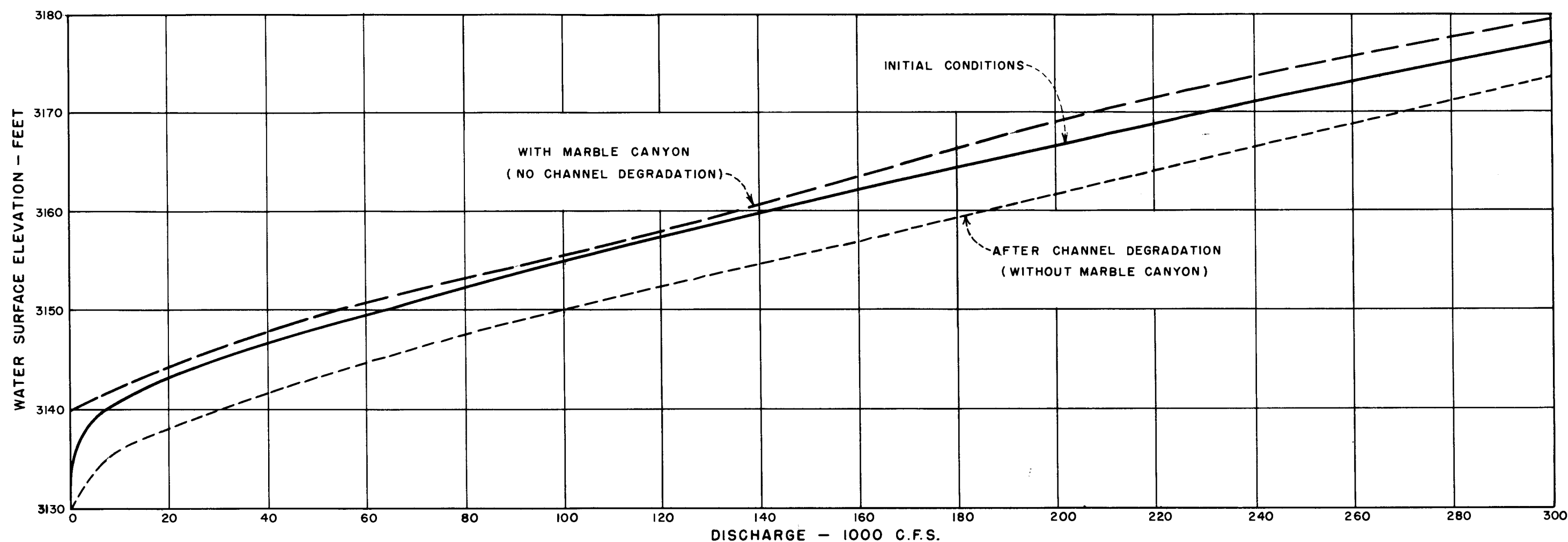
NOTES

1. Condition with Marble Canyon built
considers Marble Canyon W.S. at the
dam at elev. 3145 at 280,000 c.f.s. —
elev. 3143 at 200,000 c.f.s. and elev. 3140
at 100,000 c.f.s. and under.
2. Channel degradation based on removal
of sands overlying gravels.
3. Condition with Marble Canyon is approx.
same with as without channel degradation
Also assumes that sediment retention
will be provided on Paria River.

TAILWATER CURVES
COLORADO RIVER
R-20
(APPROX. 3400 FT. BELOW DAM AXIS)
Prepared in Hydrology Branch, Denver

12-11-56
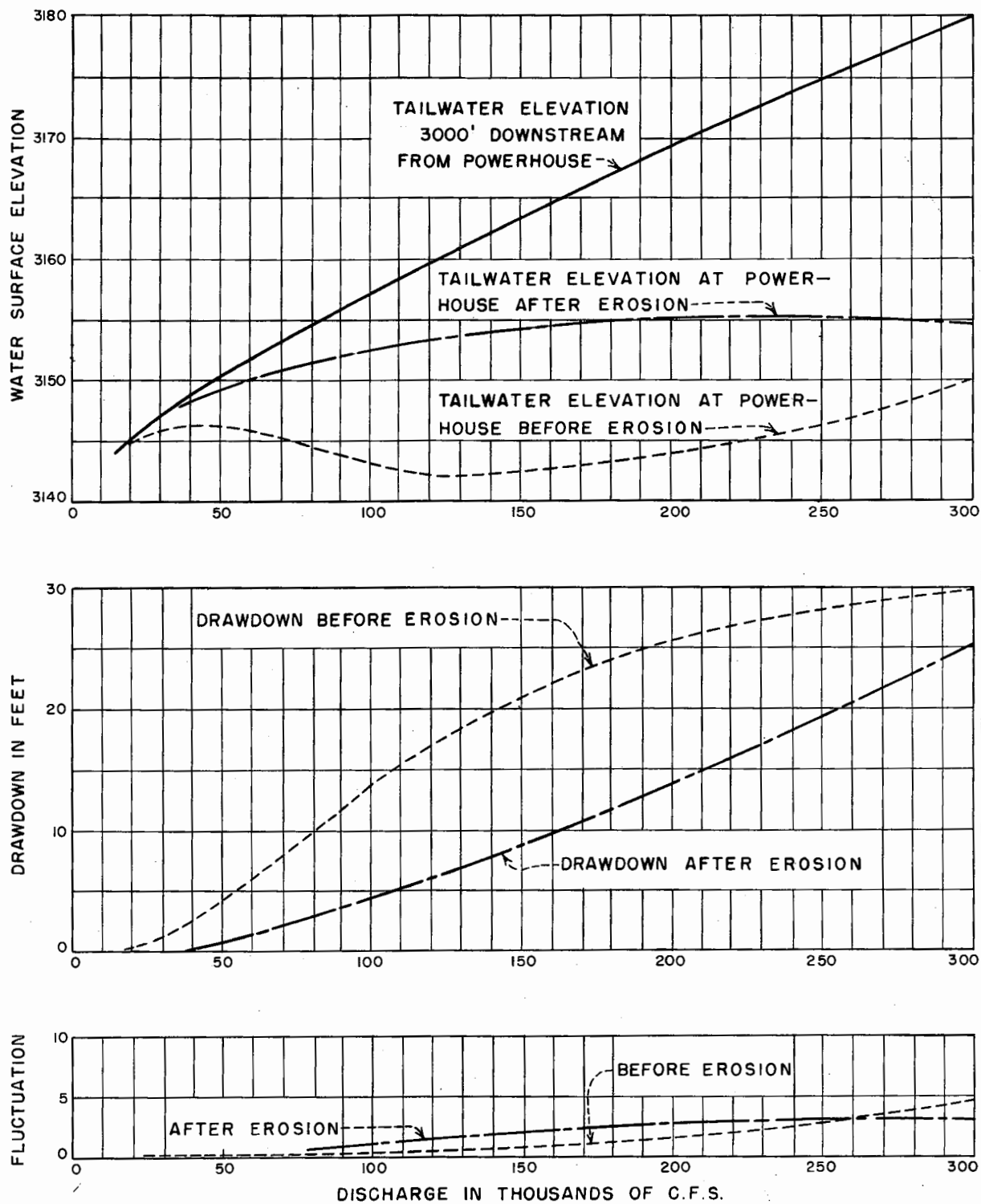
662.

FIGURE 76
REPORT HYD. 469

GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
WATER SURFACE DRAWDOWN TESTS

662

FIGURE 77
REPORT HYD. 469
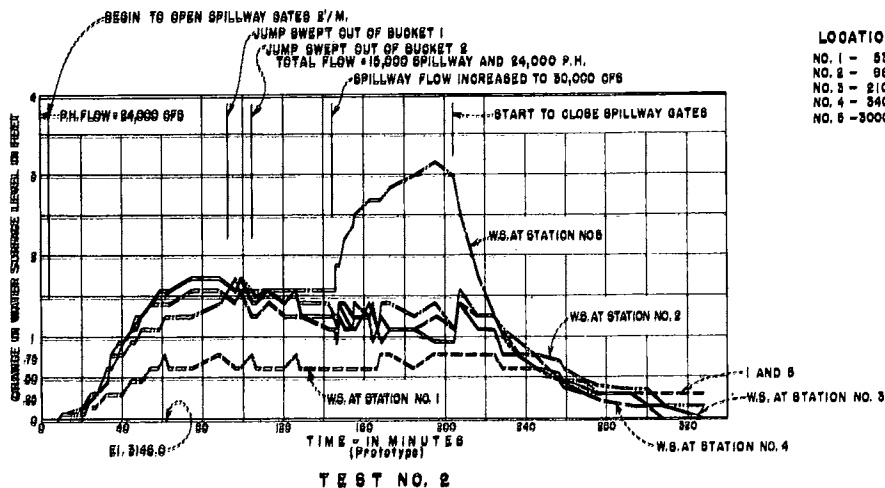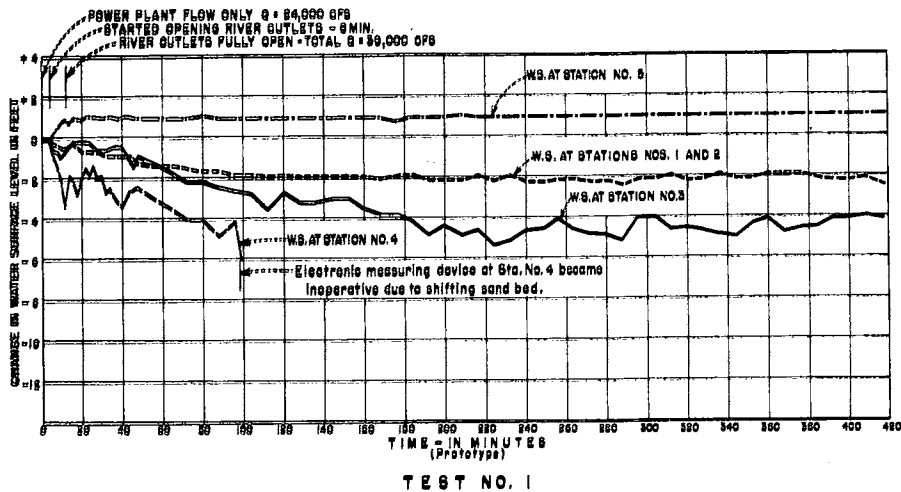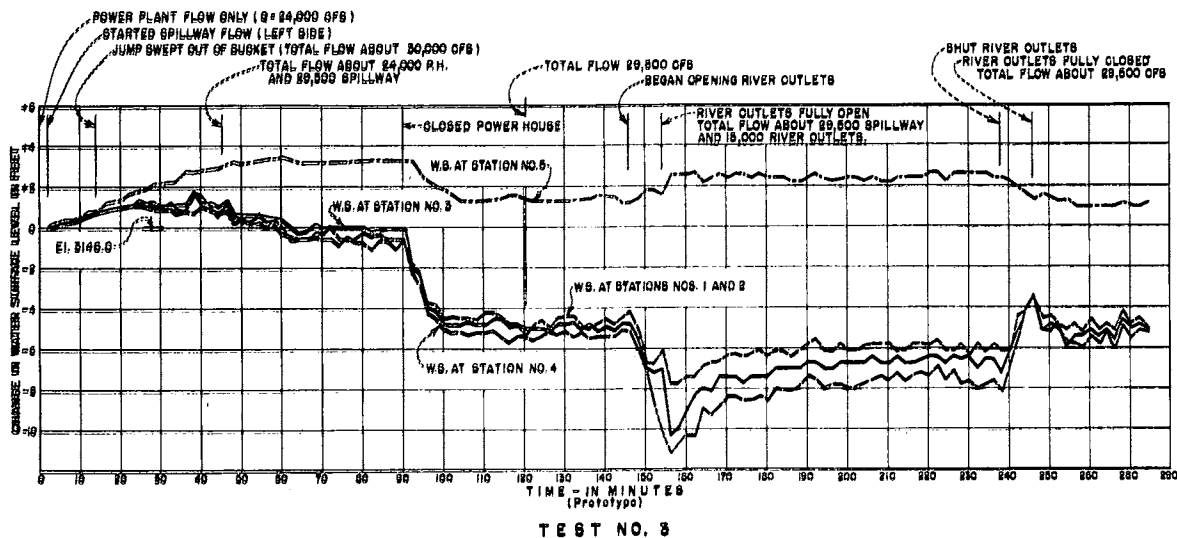
GLEN CANYON DAM SPILLWAYS
1:63.48 SCALE MODEL
WATER SURFACE DRAWDOWN
AT POWERHOUSE AFTERBAY

ABSTRACT

Hydraulic model studies were performed to investigate flow con-
ditions in the diversion works, the tunnel spillways, and the river
outlet works. The alinement of the tunnels was satisfactory for
both diversion and spillway flows. A low, curved concrete wall
placed adjacent to the right canyon wall will protect the canyon
wall from undermining and erosion damage by diversion flows.
The spillway approach channels were greatly reduced from their
original size. Flow through the crest sections was excellent and
no adverse pressure conditions were noticed. However, the pre-
liminary tunnel transition was too abrupt as indicated by rough
flow conditions and subatmospheric pressures. A longer, ade-
quately streamlined transition was developed for prototype con-
struction. Flow in the 41-foot-diameter tunnels was excellent at
all discharges. The preliminary rectangular flip bucket at each
downstream tunnel portal was replaced by a bucket in which the
circular invert of the tunnel intersected the vertical curve of the
bucket. This type of bucket eliminated the need for a circular-
to-rectangular transition. The outside walls of both buckets were
turned inward to direct the flow into the river in a more favorable
pattern. Pressures as great as 211 feet of water were measured
in the invert and on the wall of the left bucket. The river outlets
were arranged in a fan shape to reduce their erosive tendencies
in the river channel. Tailwater drawdown tests indicated that the
tailwater elevation at the powerhouse will be as much as 30 feet
lower than the downstream water level.

ABSTRACT

Hydraulic model studies were performed to investigate flow con-
ditions in the diversion works, the tunnel spillways, and the river
outlet works. The alinement of the tunnels was satisfactory for
both diversion and spillway flows. A low, curved concrete wall
placed adjacent to the right canyon wall will protect the canyon
wall from undermining and erosion damage by diversion flows.
The spillway approach channels were greatly reduced from their
original size. Flow through the crest sections was excellent and
no adverse pressure conditions were noticed. However, the pre-
liminary tunnel transition was too abrupt as indicated by rough
flow conditions and subatmospheric pressures. A longer, ade-
quately streamlined transition was developed for prototype con-
struction. Flow in the 41-foot-diameter tunnels was excellent at
all discharges. The preliminary rectangular flip bucket at each
downstream tunnel portal was replaced by a bucket in which the
circular invert of the tunnel intersected the vertical curve of the
bucket. This type of bucket eliminated the need for a circular-
to-rectangular transition. The outside walls of both buckets were
turned inward to direct the flow into the river in a more favorable
pattern. Pressures as great as 211 feet of water were measured
in the invert and on the wall of the left bucket. The river outlets
were arranged in a fan shape to reduce their erosive tendencies
in the river channel. Tailwater drawdown tests indicated that the
tailwater elevation at the powerhouse will be as much as 30 feet
lower than the downstream water level.

ABSTRACT

Hydraulic model studies were performed to investigate flow con-
ditions in the diversion works, the tunnel spillways, and the river
outlet works. The alinement of the tunnels was satisfactory for
both diversion and spillway flows. A low, curved concrete wall
placed adjacent to the right canyon wall will protect the canyon
wall from undermining and erosion damage by diversion flows.
The spillway approach channels were greatly reduced from their
original size. Flow through the crest sections was excellent and
no adverse pressure conditions were noticed. However, the pre-
liminary tunnel transition was too abrupt as indicated by rough
flow conditions and subatmospheric pressures. A longer, ade-
quately streamlined transition was developed for prototype con-
struction. Flow in the 41-foot-diameter tunnels was excellent at
all discharges. The preliminary rectangular flip bucket at each
downstream tunnel portal was replaced by a bucket in which the
circular invert of the tunnel intersected the vertical curve of the
bucket. This type of bucket eliminated the need for a circular-
to-rectangular transition. The outside walls of both buckets were
turned inward to direct the flow into the river in a more favorable
pattern. Pressures as great as 211 feet of water were measured
in the invert and on the wall of the left bucket. The river outlets
were arranged in a fan shape to reduce their erosive tendencies
in the river channel. Tailwater drawdown tests indicated that the
tailwater elevation at the powerhouse will be as much as 30 feet
lower than the downstream water level.

HYD-469
Rhone, T. J.
HYDRAULIC MODEL STUDIES OF THE SPILLWAYS AND OUTLET
WORKS--GLEN CANYON DAM--COLORADO RIVER STORAGE
PROJECT, ARIZONA
Laboratory Report, Bureau of Reclamation, Denver,
42 p, 2 tables, 39 photos, 42 figures, 1964

DESCRIPTORS--*hollow jet valves/ *radial gates/ afterbays/
diversion tunnels/ *flip buckets/ hydraulic structures/ uplift
pressures/ cavitation/ control structures/ discharge coefficients/
flow/ Froude number/ head losses/ *hydraulic models/ jets/
Manning formula/ open channel flow/ spillway crests/ surges/
tailrace/ piezometers/ pressure measuring equipment/ energy
losses/ negative pressures/ stream erosion/ backwater/ draw-
down/ transitions/ training walls/ velocity/ velocity distribution/

IDENTIFIERS--approach channel/ tunnel transitions/ tunnel
spillway