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OROVILLE DIVERSION TUNNELS 502

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

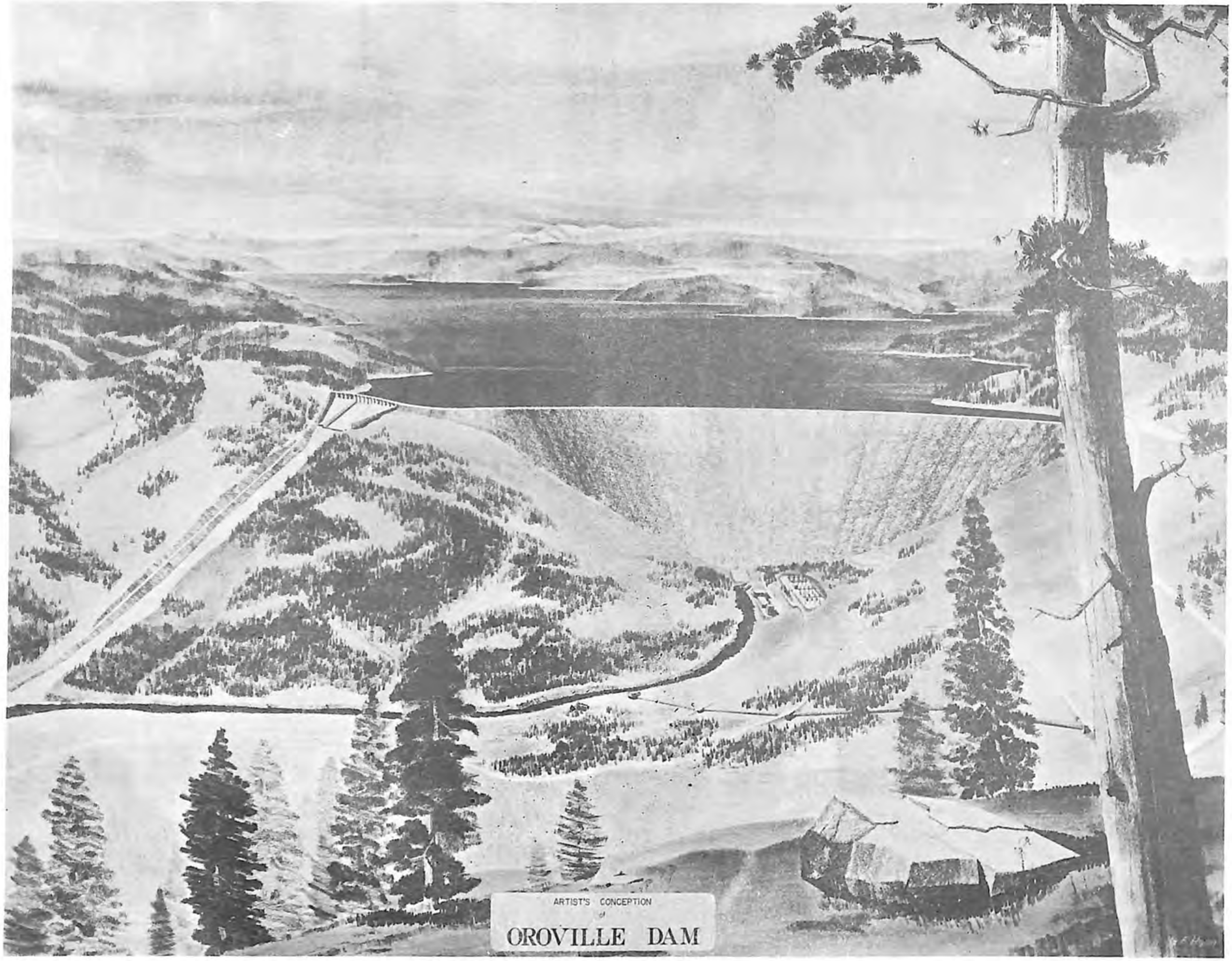
HYDRAULIC MODEL STUDIES OF THE DIVERSION TUNNELS FOR OROVILLE DAM
CALIFORNIA DEPARTMENT OF WATER RESOURCES
STATE OF CALIFORNIA

Hydraulics Branch Report No. Hyd-502

DIVISION OF RESEARCH

OFFICE OF CHIEF ENGINEER
DENVER, COLORADO

January 18, 1963


ARTIST'S CONCEPTION
of
OROVILLE DAM

PREFACE

Hydraulic model studies of features of Oroville Dam and Powerplant were conducted in the Hydraulic Laboratory in Denver, Colorado. The studies were made under Contract No. 14-06-D-3399 between the California Department of Water Resources and the Bureau of Reclamation.

The basic designs were conceived and prepared by Department of Water Resources engineers. Final designs were established through model studies that verified the adequacy of the basic designs, or led to modifications needed to obtain more satisfactory performance. The high degree of cooperation that existed between the staffs of the two organizations helped materially in speeding final results.

During the course of the studies, Messrs. H. G. Dewey, Jr., D. P. Thayer, G. W. Dukleth, J. J. Doody, and others of the California staff visited the laboratory to observe the tests and discuss model results. Mr. K. G. Bucher of the Laboratories Branch of the Department was assigned to the Bureau laboratory for training and to expedite the test program. His assignment to the laboratory materially assisted the test program. Mr. Dukleth served as Liaison Officer between the Bureau and the Department.

UNITED STATES
DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

OFFICE OF CHIEF ENGINEER

IN REPLY
REFER TO:

BUILDING 53, DENVER FEDERAL CENTER
DENVER 25, COLORADO

Mr. William E. Warne, Director
Department of Water Resources
State of California
Sacramento 2, California

Dear Mr. Warne:

I am pleased to submit Hydraulics Branch Report No. Hyd-502 which constitutes our final report on studies conducted on the diversion features of Oroville Dam. I believe you will find this report interesting and informative, and that it will satisfy the requirements of your office for a comprehensive discussion of the extensive test program.

Sincerely yours,

B. P. Bellport
Chief Engineer

Enclosure

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Office of Chief Engineer
Division of Research
Hydraulics Branch
Denver, Colorado
January 18, 1963

Report No. Hyd-502
Compiled by: W. P. Simmons
Reviewed by: W. E. Wagner
Submitted by: H. M. Martin

Subject: Hydraulic model studies of the diversion tunnels for Oroville Dam--California Department of Water Resources--State of California

PURPOSE

Studies were made to investigate the adequacy of the proposed dual purpose diversion and tailrace tunnels for carrying diversion flows past the damsite, and to determine any design changes needed to insure satisfactory operation.

CONCLUSIONS

1. The unusual vertical alinement of the tunnels, which is dictated by their eventual use as tailrace tunnels, is satisfactory for diversion purposes (Figure 3).

2. The proposed rectangular-to-circular, divided bellmouth inlet for Tunnel 1 produces good pressure and flow conditions and is satisfactory for prototype use (Figure 5).

3. The circular bellmouth inlet for Tunnel 2 also produces good pressure and flow conditions, and is satisfactory for prototype use (Figure 6).

4. Entrance losses of 0.10 $\frac{V_1^2}{2g}$ and 0.05 $\frac{V_2^2}{2g}$ occurred through Inlets 1 and 2, respectively, when flowing full under appreciable heads (Figure 15B). V_1 and V_2 are Q/A velocities in the respective tunnels. The loss in Inlet 1 rose to 0.13 $\frac{V_1^2}{2g}$ when vortices formed at flows between 30,000 and 60,000 cubic feet per second in the tunnel.

5. Large vortices form in the pool over the inlets when moderate discharges occur (Figure 16). Objects such as construction timbers, oil drums, and 40-foot trees will be readily drawn into the vortices and carried into the tunnels.

6. Air vents are not required in or downstream from the tunnel inlets.

7. Free discharge flow conditions, followed by a hydraulic jump, occur in Tunnel 1 for flows less than 29,000 cubic feet per second (Figure 17). Bedload material swept into this jump will not be continually recirculated to abrade the tunnel walls, but will be carried past the jump to work slowly out of the tunnel.

8. Part of the air entrained in the hydraulic jump collects in large bubbles that move upstream and vent at the jump face (Figures 17B and 17C). The remainder moves slowly downstream in relatively small accumulations to vent at the outlet portal (Figure 17D).

9. A 12-foot subatmospheric pressure occurs on the tunnel crown at the point where the downslope increases from 0.0028 to 0.0300 (Station 19+98) when Tunnel 1 just fills (Q = 29,000 in the tunnel). As the discharge is increased, the pressure rises. When the discharge is decreased, the flow breaks free from the crown.

10. Tunnel 2 flows full when it discharges 24,000 cubic feet per second or more.

11. The right wall of Outlet Portal 1 should be extended at elevation 232.0 for the full 50-foot length of the structure to reduce the tendency of water in the river channel to crowd the discharging water to the left (Figure 20).

12. The pressures acting on the right wall of Portal 1 are moderate and do not fluctuate in a manner likely to induce resonant conditions (Figures 20 and 21).

13. The initially proposed excavated channels downstream from the tunnel portals confined the flow. Better conditions were obtained with the excavations made wider, and with a downwardly sloped invert downstream from Tunnel 2 (Figure 22).

14. Fifteen- to eighteen-foot surges in the water surface occurred in the constricted river channel lying between the toe of the dam and the outlet portals (Figures 26A and 26B). By filling this channel with spoil material, better flow conditions were obtained in the downstream river channel (Figures 30 and 31).

15. Twenty-one- to forty-one-inch riprap will adequately protect the 2:1 sloping face of the fill, provided that graded material underlies the riprap to provide free drainage (Figure 27).

16. Spur walls extending into the stream outward from the right riverbank did not provide enough control of the clockwise eddies on the right of the outlet portals to justify their construction (Figures 23, 24, and 26).

17. Invert and sidewall deflectors at the downstream ends of the outlet portals to direct the discharging water upward or to the side, did not produce overall improvement in the river channel flow.

18. Removal of prominent rock outcrops in the downstream river channel to provide a straighter flow path did not improve flow conditions enough to justify the expense.

INTRODUCTION

Oroville Dam and its related appurtenances are a part of an immense construction job being undertaken by the State of California through its agency, The Department of Water Resources. The dam and reservoir are key features of the multipurpose Oroville Division of the Feather River Project, which is a principal part of the far-reaching California Water Plan.

The dam is being built across the Feather River at a point about 5-1/2 miles upstream from Oroville, California (Figure 1). It will be an earth and rock fill structure rising 735 feet above the riverbed, and will be the highest dam in the Western Hemisphere (Figure 2). A gate-controlled spillway located in a natural saddle on the right abutment of the dam will discharge flood waters. Power will be developed from scheduled releases of water by a 600, 000-kilowatt underground powerhouse located within the left abutment. An outlet works consisting of two 54-inch-diameter, high head valves will discharge water needed for downstream commitments after the diversion tunnels are closed and before the powerhouse releases begin, and for emergency releases at any subsequent time.

During construction of the dam, the Feather River will be diverted past the site through two 35-foot-diameter, approximately 4, 500-foot-long tunnels driven through the left abutment (Figures 2 and 3). The tunnels will be lined with concrete. After construction of the dam is completed and diversion of the river is no longer needed, the tunnels will be plugged near their midpoints and the upstream portions abandoned. The downstream portions will be used as tailrace tunnels for the underground powerplant and as a discharge tunnel for the outlet works.

Considerations for the eventual use of the tunnels as tailrace tunnels predominated in determining their vertical and horizontal alinements.

The Department of Water Resources engineers established that Tunnel 1 should act as a pressure tunnel at all times during generation. Therefore, the portion of the tunnel extending from the powerhouse to the outlet portal is located below the normal expected tailwater at elevation 225.0 (Figure 3). Four of the six powerhouse turbines will discharge into Tunnel 1.

Tunnel 2 acts as a free flow tunnel and two turbines discharge into it. Thus, Tunnel 2 is situated high enough to insure free flow at all times during power generation, and extends horizontally from the powerplant to the river channel (Figure 3). Surge port interconnections are provided from the free flow tunnel to the completely filled tunnel to allow interchange of water to avoid severe pressure surges in the filled tunnel during powerplant operation.

Problems associated with site restrictions, and with closing the tunnels after diversion, determined the elevations for the tunnel inlets. Low riverflows will be carried entirely by Tunnel 1, and its inlet invert is placed at elevation 210.0. Tunnel 2 will not operate until appreciable flow is going through Tunnel 1, and, after diversion, will be the first closed during a low water period. No special cofferdamming will be required at the inlet, and its invert is placed at elevation 230.0, 20 feet higher than Tunnel 1.

Final closure of Tunnel 1 will be made the year following closure of Tunnel 2. The flow going through the tunnel will be shut off by installing bulkhead gates across the inlet. From the time the gates are lowered until the plug is built within the tunnel, these gates must hold back the water that will begin rising in the reservoir. The only releases possible during this period will be made through the outlet works in the plug of Tunnel 2.

The high heads that may be encountered, and the 35-foot diameter of the tunnel made it impractical to use a single gate for the closure of Tunnel 1. Accordingly, a dividing pier was provided in the inlet to reduce the gate spans to about 17 feet. The necessity of the dividing pier, and of providing appropriate passage shapes at the gates, required a special inlet transition that starts as a divided rectangular opening and gradually changes to the circular section of the tunnel (Figure 5).

Since closure of Tunnel 2 does not require gates, a standard circular bellmouth inlet was satisfactory for the entrance (Figure 6).

During early construction of the dam, provisions will be made to pass floodflows that exceed the tunnel capacities over the top of the embankment. As the dam rises higher, no overtopping can be tolerated and all the flow must pass through the tunnels. On the basis of the design flood, the tunnels must be capable of discharging about 190,000 cubic feet per second. If this flood occurs, the two tunnels will flow completely full with velocities of about 100 feet per second. The temporary reservoir pool would reach a depth of nearly 400 feet.

A critical period can also occur after Tunnel 2 is plugged. This plugging is scheduled for 1966, and during the 1966-67 runoff season only the capacity of Tunnel 1 will be available. If the design flood should occur with only one tunnel operating, the outflow will be about 110,000 cubic feet per second, and the velocity 115 feet per second. The temporary pool depth would be nearly 500 feet.

The magnitudes of the discharges, flow velocities, and energy releases through the tunnels into the downstream river channel were sufficient to warrant special care in preparing final designs. Paradoxically, problems were also anticipated at very low flows due to the unusual alinement of Tunnel 1 with its downward slope to a low point and its subsequent upward slope to the outlet portal. The possibility of unstable hydraulic operation during changes from free flow to closed conduit flow conditions, and vice versa, during rising and falling flow cycles, was very real. Also, low-flow problems associated with air-entrainment, sediment movement, and the possible need of air vents were anticipated.

Hydraulic model tests offered the most practical and effective way of studying these diverse and important flow problems. After considering facilities available throughout the country, the California Department of Water Resources initiated a contract with the Bureau of Reclamation to conduct the necessary studies. This report discusses the facilities used, the tests made, and the results obtained in the program concerning diversion tunnel flows. Other reports discuss studies made on the tailrace facilities and surge conditions, outlet works, penstock inlet works and the spillway. 1/

MODEL

The 1:54.63 scale model represented an 845-foot-long section of the river channel at the entrances to the tunnels, the tunnel inlets, the two diversion tunnels, their outlet portals, and a 1,665-foot-long section of the downstream river channel (Figures 8, 9, 10, and 11). The upstream topography was contained in a sheet-metal-lined wooden box 18 feet long, 15 feet wide, and 8.5 feet deep. The box containing the downstream topography was 35.5 feet long, 12 feet wide, and 2.5 feet deep. Outlines

1/Numbers indicate references in Bibliography.

of Kelly Ridge Powerplant, a feature of the Oroville-Wyandotte Irrigation District's South Fork Project constructed below Oroville damsite, were included in the tail box.

The topography of the river channel was built up with horizontal wooden templates cut to the contour shapes of specific elevations and appropriately placed in the model (Figure 9A). These templates were covered with expanded metal lath that was stretched to conform to the ridges and valleys of the hillsides, and a 3/4-inch-thick layer of concrete was placed over the lath to produce the finished surfaces (Figure 9B). Details such as the railroad grade and U.S. Highway No. 40A were included in the head box.

The 35-foot-diameter tunnels are represented by 7.69-inch inside-diameter, transparent plastic pipe (Figure 10A). Straight pipe sections were purchased from commercial sources, and curved sections were fabricated in the laboratory shops. All sections were trimmed to appropriate length and flanged in the laboratory shops. The bends are exact model equivalents of the prototype bends.

The friction encountered by the flow in the model tunnels was greater, relatively, than that to be encountered in the prototype tunnels. To compensate for this, the four straight sections of each tunnel were shortened. The total necessary shortening was 4.9 percent of the overall tunnel lengths and was based upon obtaining conformity for open channel flow conditions. In Tunnel 1, the first, second, third, and fourth straight sections were shortened 1.6, 0.3, 0.7, and 2.2 percent of the total length, respectively. In Tunnel 2, the shortening was 1.3, 1.2, 0, and 2.3 percent. The exact shortening in each model section was dictated by requirements for maintaining correct tunnel spacings and by the need for retaining essentially the full-scale length of tunnel in the third straight section where the powerhouse connections would be made in later studies.

The tunnels were alined in plan to represent the 70-foot spacing between the tunnels in the region of the powerhouse, as shown in early drawings. The spacing was later changed to 80 feet, but the diversion model was not altered because no significant flow difference would result. The alinement in elevation was that specified for the final design (Figure 3).

The entrance to Tunnel 1 is a gradual transition from the rectangular inlet to the circular tunnel (Figure 5). The entrance is flared on both sides and at the top with elliptical curves having semiminor axes one-third the length of the semimajor axes. 5/ The semimajor axis for the top curve equals the conduit diameter; the axis for the side curves approximately equals the width of the passages between the outside walls and the center pier. The semimajor axis of the elliptical curves on the pier approximately equals the passage width, and is five times longer than the semiminor axis. 6/ All curvature of the sides and center pier, and most of the curvature of the top, is completed before the transition from the rectangular to the circular section begins. The downstream edges of the gate slots in the sidewalls, roof, and center pier are offset outward from the flow and are followed by curved surfaces that return the passages to the normal transition boundaries. This design has previously been found desirable for avoiding cavitation damage. 7/

The complexity of this inlet structure made the use of transparent plastic the most practicable method for constructing and viewing the flow in the model. Wooden patterns were accurately shaped, and heated 1/8-inch-thick plastic sheets were formed over them and allowed to cool. The shaped plastic sections were trimmed of excess material, carefully fitted together, and cemented. Appropriate flanges were fitted on either end, and reinforcing fillets were cemented over all the joints. Piezometers were placed at strategic points in the transition to permit studying pressure conditions on the interior surfaces (Figures 5 and 11A).

The circular bellmouth inlet for Tunnel 2 was less complicated and was made by screeding a dense concrete mix into a metal container that was flanged to attach to the circular tunnel (Figures 6 and 11B). The bellmouth entrance was formed by an elliptical curve with a semimajor axis equal to the conduit radius and a semiminor axis equal to 0.3 of the conduit radius. Piezometers were installed by fastening 1/16-inch inside-diameter brass tubes onto metal ribs that followed the elliptical curve. The tubes extended to the surface of the finished concrete and were perpendicular to it. The other ends of the tubes extended through the container and were connected by flexible plastic tubing to water manometers.

Excavations and the positions and slopes of the outlet portals at the beginning of the test program conformed to the designs shown in early drawings (Figure 12). During the test program it was found desirable to enlarge the excavations, and these final configurations are shown on later drawings.

Water was supplied to the model through the central laboratory water-supply system. Rates of flow were measured by permanently installed, volumetrically calibrated Venturi meters. Water entered the head box in the space upstream from the rock baffle (Figure 8). After passing through the baffle, the flow entered the upstream river channel free of large-scale turbulences. The water passed through the head box, tunnels, and tail box of the model, and then returned to the laboratory reservoir for recirculation. A tailgate at the end of the tail box allowed setting and maintaining various water surface elevations.

Pressures on the flow surfaces of the inlet, tunnels, and outlet structures were measured with water-filled manometers. The pressures at selected critical regions were also measured by means of pressure cells coupled to multichannel recorders. Wave heights in the downstream river channel were measured by staff gages and by a capacitance-type wire depthometer coupled to a recorder. Head loss measurements across the inlets were made with water manometers.

Just before the model tests were started, a rockfall occurred at the upstream portal where Diversion Tunnel 1 was being excavated into the hillside at the project. A revised tunnel portal support system was designed to restore normal service over the Western Pacific Railroad, located immediately above the tunnel portal. The revised portal support was placed forward of the tunnel face as a cut-and-cover section and filled over. This necessitated displacing the inlet transition 40 feet farther upstream, and a corresponding change was made in the model (Figure 13).

INVESTIGATION

Inlet Structures

Detailed pressure measurements were made on the flow surfaces of the inlet structures of Tunnels 1 and 2 (Figures 14 and 15). Measurements were made with Tunnel 1 operating alone, and with Tunnels 1 and 2 operating together. Test discharges ranged from 5,000 cubic feet per second up to the near maximum of 190,000 cubic feet per second. Single-leg, water-filled manometers were used for the measurements, and electronic pressure cells were used to further check the few pressures found to be subatmospheric, or to fluctuate widely. The data are presented as pressure head, in feet of water, prototype, that will act at the location of each piezometer. In general, the pressures were entirely satisfactory. A few subatmospheric pressures were found at small discharges when the water surface was at or near the piezometer openings. These pressures, as determined by pressure cell measurements, did not exceed subatmospheric values of 5 or 6 feet of water, prototype, and will not be troublesome.

Increases in rate of flow caused corresponding increases in piezometric pressures because back pressure was developed by friction in the long tunnels.

The friction in the model tunnels flowing full was slightly greater than the equivalent expected in the prototype tunnels flowing full. This greater friction loss produced slightly higher equivalent pressures at the model inlets than those which will occur on the prototype at equivalent discharges. This factor is of little importance at small flows, but may be appreciable at high flows. No compensation, other than the previously mentioned tunnel foreshortening, was provided in the presented data because of the uncertain nature of the actual prototype friction, and because the pressures were strongly positive and safe. A correction could be applied by reducing the pressures linearly in accordance with any anticipated reduction in the friction head.

At large discharges, and particularly with both tunnels operating, pressure fluctuations were found along the top flow surface near the entrance of Tunnel 1 (Figure 14). The water surface fluctuations occurred between two fairly definite upper and lower levels and indicated that two types of flow regimes were being established. No difficulty is expected from these conditions because the fluctuations were limited to the corner areas where great structural rigidity will be available and because they exhibited no regular period that could excite a resonant vibration frequency.

Strong and persistent vortices occurred above the inlets in the reservoir pool at moderate discharges (Figure 16). Their positions were not fixed, but tended to wander. At low submergences the vortices were always present, whereas at submergences greater than about 100 feet they sometimes disappeared, to re-form and reappear later. Their power was manifested by their ability to "swallow" large objects equivalent to 50-gallon oil drums and trunks of 30- and 40-foot trees. Occasionally, they were accompanied by loud gurgling and sucking noises. The "ropes" of air admitted into the tunnels by the vortices broke up into bubbles that swept directly through the system without causing difficulties.

Air vents into the tunnels at, and just downstream from the inlet transitions, did not effect the inception or the growth of the vortices because the tunnel pressures were positive and no air was drawn through the vents when the inlets were submerged. The vents were of no value during unsubmerged operation, and are not recommended for the field structure.

The head losses through the bellmouth inlets were low. A loss of $0.10 \frac{V^2}{2g}$ was measured in the rectangular, divided bellmouth of Tunnel 1 with both single-tunnel and two-tunnel operation at large discharges (Figure 15B). At flows below 60,000 cubic feet per second in the tunnel when large vortices were present, the loss factor rose to 0.13. These losses were measured from the reservoir water surface to the energy gradeline in the tunnel 9 feet 11 inches, prototype, downstream from the end of the curved entrance (Piezometers 55 and 56, Figure 5). A loss of 0.05 $\frac{V^2}{2g}$ (Figure 15B) occurred through the circular inlet of Tunnel 2 to a station 48.6 feet downstream from the tunnel inlet (Piezometer 36, Figure 6).

Tunnels

Flow in the tunnels ranged from the free discharge conditions that prevail during low flows, through a transition range at higher discharges, to full pressure flows at large discharges.

Tunnel 1.--Unusual flow conditions occurred in Tunnel 1 due to the downward slope to Station 32+70, followed by the upward slope to the outlet portal (Figure 3). A free water surface occurred throughout the tunnel at flows from 0 to about 9,000 cubic feet per second, with tailwater elevations as shown in Figure 18B. At a flow between 9,000 and 10,000 cubic feet per second the tunnel filled at the low point (Station 32+70), and gradually filled toward the outlet portal as the discharge increased to about 16,000 cubic feet per second.

At flows below 21,000 cubic feet per second, free discharge conditions prevailed from the tunnel inlet to the beginning of the third horizontal bend (Station 27+57). The flow was subcritical in the section from the inlet to Station 19+98 where the slope is 0.0028. At Station 19+98 the slope increases to 0.0300 and supercritical flow was established. This supercritical flow plunged into the full or partially full region at Station 27+57 and formed a hydraulic jump (Figures 17A, 17B, and 17C).

Air entrained in the hydraulic jump at Station 27+57 rose and moved downstream to gather into one or two large bubbles at the top of the tunnel. Intermittently the upstream bubble, due to its buoyancy and the favorable slope of the tunnel, moved upstream and vented itself at the face of the jump (Figures 17B and 17C). The second bubble, when present, was farther downstream (Station 30+00) and more stable. It persisted for long periods in a more or less fixed position with air entering at the upstream end and with bubbles leaving at the downstream end (Figure 17D).

At discharges approaching 22,000 cubic feet per second, the upstream part of Tunnel 1 flowed nearly full. Standing waves with trough-to-crest heights of about 4 feet produced a sinuous water surface through the tunnel to Station 19+98. Beyond this point, supercritical flow with a smooth water surface was established.

At about 24,000 cubic feet per second, Tunnel 1 filled near the bellmouth inlet. Once this filling occurred, the tunnel filled slowly and progressively downstream to Station 19+98. Beyond Station 19+98, supercritical flow persisted, and the hydraulic jump continued to occur near the third horizontal bend. Air entrained by the jump continued to collect in bubbles that vented upstream into the jump area, and in bubbles that moved toward the tunnel outlet and vented into the atmosphere.

Two interesting phenomena were observed in the model with the above flow conditions. In the first, the station at which the flow passed through critical depth remained fixed near Station 19+98. Then, as air was evacuated from the tunnel by the jump, the jump moved upstream. This lowered the ambient pressure into which the filled, upstream portion of the tunnel discharged, and increased the rate of flow from the reservoir. The greater flow withdrawn from the model reservoir (head box) lowered the water surface so that ultimately air was drawn into the tunnel, and the tunnel from the inlet to Station 19+98 ceased to flow full. The discharge under the open channel flow conditions decreased, and the smaller flows allowed the reservoir water surface to rise so that conditions were again established to cause the tunnel to flow full. This cycle slowly repeated as long as the flow rate into the head box was maintained constant. No undue disturbance occurred in the system during the cycles.

The second phenomenon was similar to the first, except that the station where critical depth occurred was no longer fixed. Instead, as air was entrained and evacuated from the tunnel, the point where critical depth occurred moved downstream from Station 19+98 and into the more steeply sloping pipe. The position of the jump remained about stationary. The siphonic action produced by the sloping, filled section of tunnel increased the rate of flow from the reservoir to cause the cycling described above. This phenomenon differed from the first in that the point where critical depth occurred moved down the slope; whereas, in the first case, the jump moved up the slope.

These cycling conditions are not likely to occur in the prototype, because an appropriate and nearly constant reservoir inflow over a relatively long period of time is required. If cycling should occur under appropriate field conditions, however, the cycling period will be correspondingly longer in the prototype since only a small portion of the reservoir capacity was represented in the model.

At flows of about 29,000 cubic feet per second and above, Tunnel 1 flowed full throughout its length.

When Tunnel 1 just filled, a 12-foot subatmospheric pressure occurred at the crown just downstream from Station 19+98. This pressure rose and became positive as the rate of flow increased. No difficulty is expected with the subatmospheric pressures provided the joint is made reasonably smooth, because flow velocities are low at these discharge conditions, and the crown of the tunnel slopes into the flow just past the joint. The flow is expected to break free from the crown when the discharge falls below 29,000 cubic feet per second, and open-channel flow will be reestablished.

Sand approximating bedload material from the river was introduced into Tunnel 1 during free discharge operation. The material moved readily through the hydraulic jump area and was either deposited along the invert farther downstream or migrated slowly to the outlet portals by means of traveling dunes. Scour caused by bedload recirculation in the jump area of the prototype tunnel should therefore not be severe.

Tunnel 2.--Flow conditions in Tunnel 2 were quite different. The upstream portion of the tunnel is on a 0.0136 downward slope, and at low discharges supercritical flows occurred. Farther downstream, and for the rest of the tunnel length, the tunnel slope is zero. The backwater created by the long horizontal tunnel caused a hydraulic jump to form at the start of the second horizontal curve (Station 14+33). The entire tunnel flowed full at a discharge of about 24,000 cubic feet per second. No undue disturbance occurred in the tunnel at any time.

Discharge Capacity of Tunnels

The relationship of discharge vs reservoir elevation for Tunnel 1 or Tunnel 2 operating alone, and for both tunnels operating together was determined (Figure 18A). The model data are expected to be accurate at the lower discharges because the model tunnel lengths were set to produce the equivalent frictional resistance predicted for free discharge in the prototype tunnels. When the tunnels flow full, and particularly when velocities are high, the model friction is probably greater than the equivalent prototype friction. This results in higher model reservoir elevations, for given discharges, than will be required in the prototype. Interpretation of the attached curves must be made with this factor in mind. No corrections were attempted because of the uncertain nature of the prototype loss values. The effects of approach conditions to the tunnel inlets, and of vortices that form over the structures, are included in the model data.

Outlet Portals

Initial tests were made with the portal geometries shown in Figure 12. Flow conditions were relatively quiet at small discharges, but became rough at larger flows. A large clockwise eddy occurred in the upstream part of the river channel and tended to deflect the flow from Tunnel 1 toward the left riverbank (Figure 19). By extending the right wall of Portal 1 at its full height (elevation 232.0) to the end of the concrete section (Station 45+76.60)(Figure 20) outlet flow conditions were improved. This change, which will facilitate handling the stoplogs and provide easier entry into the tunnel for plugging and other operations, was recommended for the prototype outlet.

The hydraulic loading on the extended wall was determined by piezometric pressure measurements on the inside and outside wall surfaces (Figure 20). Water-filled manometers were used to determine the average pressure conditions, and pressure cells and a multichannel recorder were used to determine instantaneous conditions at piezometers that showed the lowest pressures and/or greatest fluctuation (Figures 20 and 21). Data were taken at the maximum discharge of 190,000 cubic feet per second because the largest flows and highest tailwaters produced the greatest loads on the wall. In addition to tests at the normal tailwater elevation 250.0, tests were also made with the tailwater at elevation 255.0. Measurements showed that the loadings were not excessive and that the fluctuations did not occur with a regular period that could create a resonant condition.

The pressure tests were made with and without spur walls in place on the right bank, and with Fills 1 and 2 in the river channel. These items are discussed later in the report.

Portal 2 is located high relative to the tailwater, and changes in tailwater have no effect upon the flow depth at the portal. Thus, the right guide wall downstream from the portal is higher than necessary for satisfactory hydraulic performance, and can be lowered 7 feet to elevation 243.0 (Figure 22). No pressure measurements were necessary on this outlet portal.

The excavated rock channel initially proposed downstream from the concrete-lined sections confined the flows and subjected the rock to high velocities (Figure 12). The model channel downstream from Portal 1 was widened the equivalent of 5 feet on either side, and the channel of Portal 2 was widened the equivalent of 7 feet on either side (Figure 22). The wider channels produced more satisfactory flow conditions and the rock walls were subjected to less flow impact, and hence to less erosion.

Flow in River Channel

The locations of the outlet portals on the left riverbank and the angles at which the tunnels discharged into the river caused a large clockwise eddy to form between the jets and the right bank. At moderate and high discharges, flow velocities of about 15 feet per second and waves 6 to 12 feet high swept upstream along the right riverbank and swung across the flow discharging from Tunnel 1 (Figure 24). In addition, tide-like surges 15 to 18 feet high occurred in the channel upstream from the portals on the fill slopes of the dam and roadway (Figures 26A and 26B). Heavy splashing and severe erosion of the fill slopes were probable on the prototype structure.

Modifications to Outlet Portals

Invert deflectors.--Attempts were made to improve flow conditions in the river channel by redirecting the flow emerging from the outlet portals. The inverts of the portals were first changed by installing upslopes that produced flip or ski-jump buckets that deflected the flows upward. These invert deflectors were triangular in cross section with rises of 3 to 16 feet. Reasonable results were obtained with the larger deflectors at large discharges with single-tunnel operation, but conditions were not significantly improved for two-tunnel operation. Further studies showed that considerably higher reservoir elevations were required to pass low flows over the elevated crests of the flip buckets. Also, after diversion was completed, it would be necessary to remove the barrier formed by the invert deflectors so the tunnels could operate efficiently as tailrace tunnels for the underground power station. Tests were therefore terminated on the invert deflectors.

Other tests were made using wedge-shaped deflectors on the walls of the outlet portals to deflect the jets laterally. Considerable experimentation showed that no consistent improvements in the river channel flow were obtained by redirecting the jets. A design satisfactory for one set of flow conditions would usually be unsatisfactory for another.

Limited tests using both the sidewall and invert deflectors showed that no general improvement could be obtained, and the deflector tests were terminated.

Spur walls on right bank.--Tests were also made with spur walls that extended outward from the right bank to intercept the eddy (Figures 23 and 24). Design 2, with a position normal to the riverbank, performed best and produced a moderate reduction in the surging and waves at the right wall of Outlet Portal 1 (Figures 24 and 25). The best length for the wall was about 75 feet, with the top at elevation 226.

The pressure variations and hydraulic loadings on the right guide wall of the outlet portal were not appreciably changed by installing or removing the spur wall (Figure 21).

Instantaneous pressure readings at selected points on the spur wall showed that heavy shock loads were imposed by waves and currents. The tests indicated that the limited beneficial effect of even the best spur wall was not enough to justify the cost and inconvenience of building it, and the spur walls were not recommended for use.

Fills near outlet portals.--The river channel extending upstream from Outlet Portal 1 to the toe of the dam becomes progressively narrower due to the presence of the fill supporting the access road to the portal (Figures 26A and 26B). Waves emanating in the main river channel traveled up this restricted channel and produced trough-to-crest surges 15 to 18 feet high at the toe of the dam. These surges created severe erosion problems at the toe of the dam and on the roadway slopes and reflected back to contribute to the waves at the outlet portals.

By filling the channel with spoil material obtainable from tunneling or stripping operations, the problem was alleviated (Figures 23 and 24). Best conditions occurred with Fill 2 (with Spur Wall 2 in place). A more practical placement, and one that did not necessarily involve a spur wall, was found in Fill 1 where the toe of the fill lies at the right-hand downstream corner of Outlet Portal 1. Fill 3 was too small to be effective. A slope of 2:1 was found best for the faces exposed to the water, and the minimum height of the fills should be elevation 230 to provide protection for discharges up to at least 130,000 cubic feet per second. Fill 1 was recommended for prototype use.

Protection of fill slope.--Tests were conducted on a vertical wall that extended across the river channel from the right guide wall of Outlet Portal 1 to the right-hand bank (Figure 26C). This wall offered excellent protection to the fills near the portal and obviated the need for riprap protection on the exposed face. A wall extending downstream at a 45° angle to the guide wall was found unnecessarily long. It also confined the clockwise return eddy and forced the jet from Portal 1 into the rock underlying Portal 2. Better results were obtained by moving the wall 25 feet upstream from the downstream end of the training wall and extending it at a 70° angle across the channel to a prominent rock outcropping on the right bank (Figure 26C). Various heights of wall were investigated, and a top elevation of 220 seemed most desirable. Moderate overtopping occurred at the largest two-tunnel discharges. A relatively small amount of 18-inch riprap appeared adequate to protect fills lying above this elevation for the highest flows. The wall, however, would be difficult and expensive to build. Studies then were made to determine the effectiveness and size of riprap needed to protect the sloped face on the fill.

In the model, the face of the fill was represented by subangular gravel that passed a 3/8-inch sieve and was retained on a No. 4 sieve (Figure 27A). The face of the fill was placed on a 2:1 slope and on a bearing of approximately N 26° W. The toe was located about 10 feet downstream from the end of Outlet Portal 1.

Velocity measurements were made along the face of the fill with a miniature propeller meter that swept an area three-eighths inch in diameter (Figure 28). The hub was held three-fourths of an inch from the rock face (3.4 feet prototype) and readings were made by a counter that totalized over 10-second (model) periods. The average long-term velocities and maximum 10-second average velocities were measured at nine measuring stations for discharges of 50,000 and 81,500 cubic feet per second through Tunnel 1 and 90,000 and 135,000 cubic feet per second through both tunnels.

The first tests were made with no additional protection on the No. 4 to 3/8-inch gravel slopes. This gravel was equivalent to prototype rock sizes of 13 to 21 inches, by geometric scaling. With both tunnels operating to produce discharges of 90,000 and 135,000 cubic feet per second, moderate slumping and movement of the gravel face occurred, primarily due to wave action. With Tunnel 1 operating by itself at discharges of 50,000 and 81,500 cubic feet per second, considerable movement occurred due to high velocity flows moving along the toe toward the tunnel portal (Figure 27B).

A riprap blanket about 2 inches thick in the model, and composed of subangular gravel passing a 3/4-inch sieve but retained on a 3/8-inch sieve, was placed to grade on the fill slopes (Figure 27C). No significant movement of this heavier material occurred with two tunnels operating, and only a few pieces near the toe of the slope were displaced with single-tunnel operation (Figure 27D). These pieces were displaced by the relatively high velocity, shallow flows that swept toward the portal at discharges of 80,000 cubic feet per second or more. The direction of these flows is indicated by the arrow in Figure 27C.

The results indicate that prototype rock equivalent in size to the 3/8- to 3/4-inch subangular rock of the model should be provided to resist scour on the fill slopes. By geometric scaling from the 1:54.63 scale model, the prototype rock would be 21 to 41 inches in diameter, and should be angular or subangular in shape. The largest size rock should be concentrated at the toe of the slope to resist the shallow, high-velocity flows that occur with high discharges through one tunnel.

An interesting comparison is noted between the above riprap test results and those presented in USBR Report No. Hyd-409. 8/ The average flow velocity along the face of the rock in the Oroville model for a discharge of 81,500 cubic feet per second was equivalent to a prototype velocity of 13.4 feet per second. This velocity, according to the graph of Figure 11, Report No. Hyd-409, requires riprap with a minimum size of 26 inches. This agrees well with the 21- to 41-inch size range established above.

Channel straightening.--Rock outcrops in the downstream river channel interfered with the flow of water and appeared to aggravate the size and intensity of eddies on the right and, to a lesser extent, on the left side of the discharges from the tunnels. To alleviate these apparent conditions, the model channel was straightened by removing all or part of the outcrops (Figures 29A and 29B). Removable concrete topographic inserts were made which could be placed in the model to restore the channel to its initial contours. These inserts permitted quick and accurate appraisals of the effects of any particular excavation.

Studies showed that the flow in the downstream portion of the river channel could be appreciably improved, but that conditions near the outlet portals were only moderately affected. The greatest flow improvement near the portals was noted when the outcrop on the right bank about 700 feet downstream from Portal 1 was removed. However, it appeared that the expense of these major channel improvements was not justified and no recommendation was made for them.

Recommended Design of Outlet Portals and Downstream Channel

The recommended design for the outlet portals and downstream channel includes the full-length high wall on Portal 1, the enlarged rock channels downstream from both portals, Fill 1 in the river channel upstream from the portals, a 2:1 slope on the face of the fill, and about 36-inch-diameter riprap underlayed with pervious material to protect the fill. Flow conditions will be acceptable at all discharges up to 100,000 cubic feet per second with this design (Figure 30). At larger flows, moderate damage to the river channel downstream may occur, but no appreciable damage is expected at or near the outlet portals (Figure 31).

BIBLIOGRAPHY

1. "Hydraulic Model Studies of the Draft Tube Connections and Surge Characteristics of the Tailrace Tunnels for Oroville Powerplant," Report No. Hyd-507, by W. P. Simmons

2. "Hydraulic Model Studies of the Outlet Works for Oroville Dam," Report No. Hyd-508, by Donald Colgate

3. "Hydraulic Model Studies of the Penstock Intake Control Structure for Oroville Dam," Report No. Hyd-509, by Donald Colgate

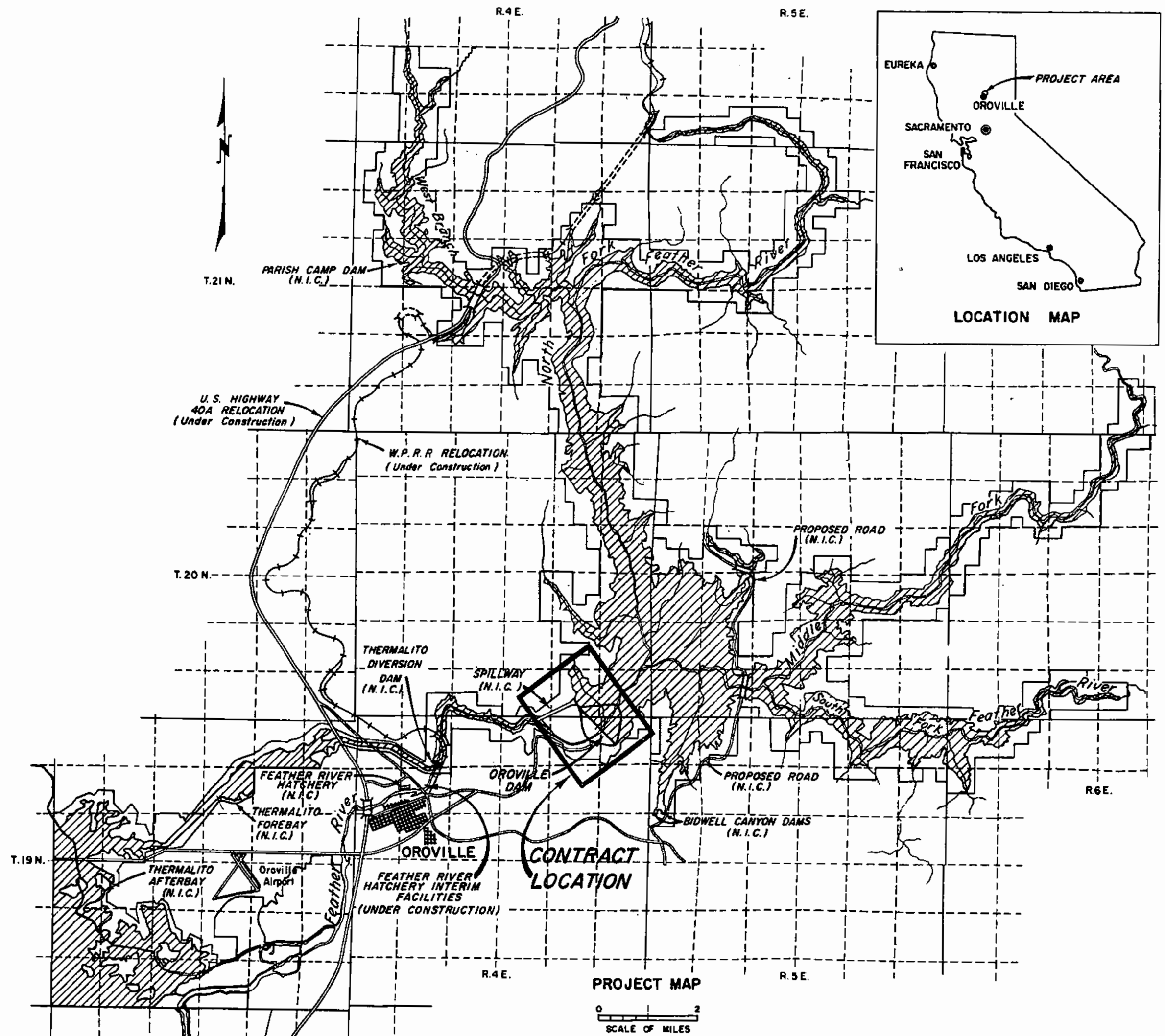
4. "Hydraulic Model Studies of the Spillway for Oroville Dam," Report No. Hyd-510, by T. J. Rhone

5. "Investigation of Entrances Flared in Three Directions and in One Direction," Technical Memorandum 2-428, June 1959, Corps of Engineers

6. "Hydraulic Model Studies of Twitchell (Vaquero) Dam Outlet Works," Santa Maria Project, California, Report No. Hyd-449, by Donald Colgate

7. "Hydraulic Characteristics of Gate Slots," by J. W. Ball, Paper No. 2224, Journal, Hydraulics Division, American Society of Civil Engineers, October 1959

8. "Stilling Basin Performance Studies--An Aid in Determining Riprap Sizes," Report No. Hyd-409, by A. J. Peterka



SUBMITTED:
CHIEF, DAM DESIGN SECTION

APPROVAL RECOMMENDED:
CHIEF, DAMS AND PLANTS DESIGN BRANCH

APPROVAL RECOMMENDED:
DEPUTY DIVISION ENGINEER

APPROVED: DATE: MAR 17 1962
DIVISION ENGINEER

APPROVED: DATE: MAR 17 1962
CHIEF ENGINEER, REG. CIVIL ENGINEER NO. 12676

DEPARTMENT OF WATER RESOURCES STATE OF CALIFORNIA

OROVILLE DAM
DIVERSION TUNNEL STUDIES
LOCATION MAP

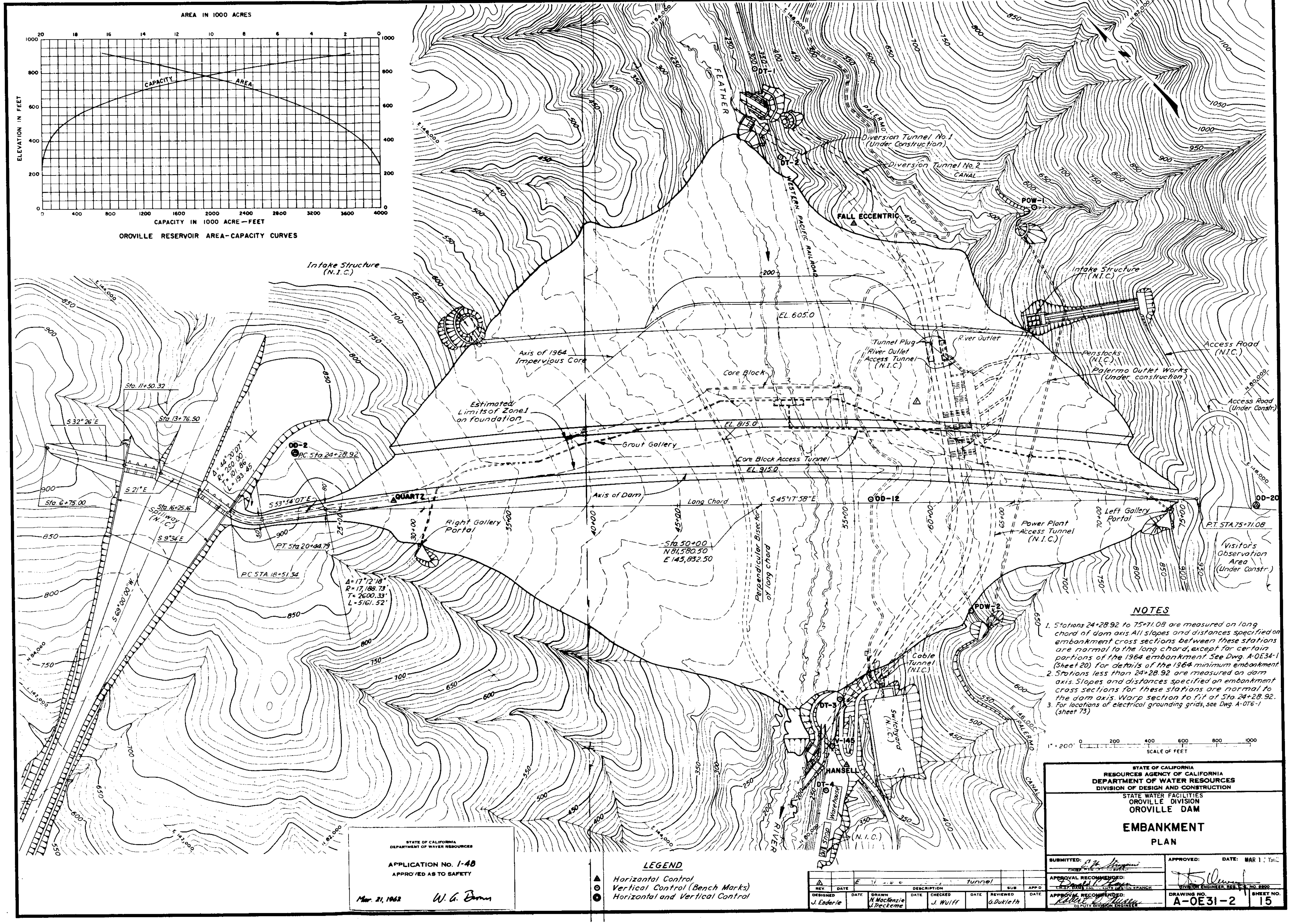
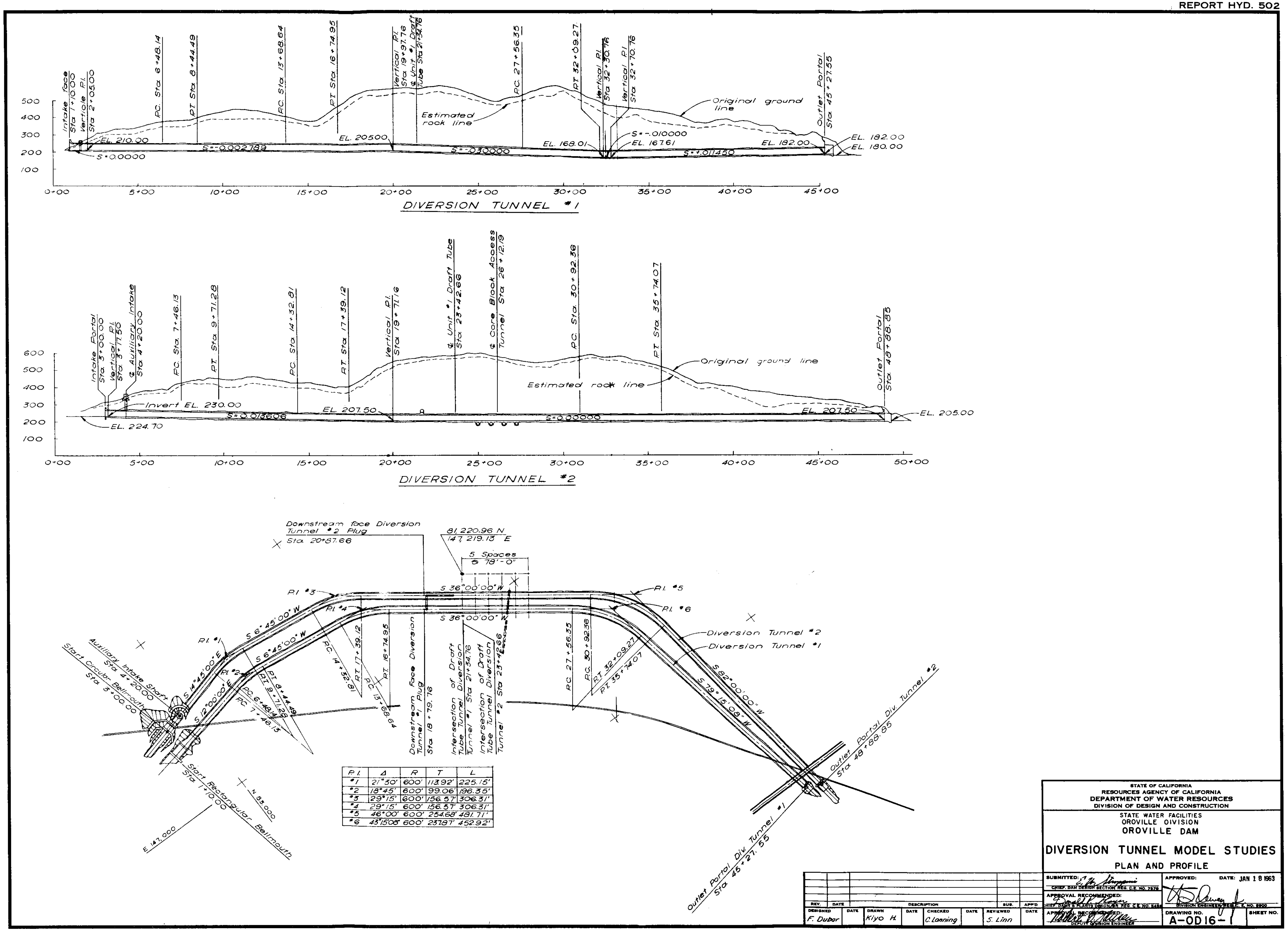

AREA IN 1000 ACRES
ELEVATION IN FEET
CAPACITY
AREA
CAPACITY IN 1000 ACRE—FEET
OROVILLE RESERVOIR AREA-CAPACITY CURVES
FEATHER
Diversion Tunnel No.1 (Under Construction)
Diversion Tunnel No.2
FALL ECCENTRIC
Intake Structure (N.I.C.)
EL. 605.0
Axis of 1964 Impervious Core
Tunnel Plug
River Outlet
River Outlet Access Tunnel (N.I.C.)
Penstocks (N.I.C.)
Palermo Outlet Works (Under construction)
Access Road (N.I.C.)
Access Road (Under Constr.)
Core Block
Estimated Limits of Zone 1 on foundation
EL. 815.0
Grout Gallery
Core Block Access Tunnel
EL. 915.0
QUARTZ
Axis of Dam
Long Chord
S 45°17'58"E
OD-12
OD-2
PC Sta 24+28.92
Sta. 11+50.32
Sta 13+76.50
S 32°26'E
S 21°E
Sta 6+75.00
Sta 16+25.16
Spillway (N.I.C.)
S 9°34'E
S 53°54'07"E
PT Sta 20+44.79
PC STA 18+51.34
Right Gallery Portal
Sta 50+00
N 81,580.50
E 143,852.50
Perpendicular Bisector of long chord
Power Plant Access Tunnel (N.I.C.)
Left Gallery Portal
P.T. STA 75+71.08
Visitors Observation Area (Under Constr.)
OD-20
POW-1
POW-2
Cable Tunnel (N.I.C.)
Switchyard (N.I.C.)
DT-1
DT-2
DT-3
DT-4
HANSELL
RIVER
PALERMO CANAL
NOTES
1. Stations 24+28.92 to 75+71.08 are measured on long chord of dam axis. All slopes and distances specified on embankment cross sections between these stations are normal to the long chord, except for certain portions of the 1964 embankment. See Dwg. A-0E34-1 (Sheet 20) for details of the 1964 minimum embankment.
2. Stations less than 24+28.92 are measured on dam axis. Slopes and distances specified on embankment cross sections for these stations are normal to the dam axis. Warp section to fit at Sta. 24+28.92.
3. For locations of electrical grounding grids, see Dwg. A-0T6-1 (sheet 73)
1" = 200'
SCALE OF FEET
STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
APPLICATION No. 1-48
APPROVED AS TO SAFETY
Mar. 21, 1962
LEGEND
Horizontal Control
Vertical Control (Bench Marks)
Horizontal and Vertical Control
DESIGNED J. Enderle
DRAWN H. MacKenzie J. Treckeme
CHECKED J. Wulff
REVIEWED G. Dukleth
STATE OF CALIFORNIA
RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION
STATE WATER FACILITIES
OROVILLE DIVISION
OROVILLE DAM
EMBANKMENT
PLAN
SUBMITTED:
APPROVED:
DATE:
APPROVAL RECOMMENDED:
DEPUTY DIVISION ENGINEER
DRAWING NO.
A-0E31-2
SHEET NO.
15

FIGURE 3
REPORT HYD. 502



P.I.	Δ	R	T	L
#1	21°50'	600'	113.92'	225.15'
#2	18°45'	600'	99.06'	196.35'
#3	29°15'	600'	156.57'	306.31'
#4	29°15'	600'	156.57'	306.31'
#5	46°00'	600'	254.68'	481.71'
#6	43°15'08"	600'	237.87'	452.92'

PLAN
Scale: 1" = 20'-0"

NOTES:

1. For portal concrete details see Dwg. A-0D10-6 (Sheet 81)
2. For auxiliary intake see Dwg. A-0D10-9 (Sheet 84)
3. Minimum portal rock bolting: 15' rock bolts on 5' square pattern on 0.5:1 and steeper slopes.
4. Chain link fabric shall be used wherever slopes are rock bolted. Headers shall be used as directed.
5. Open cut excavation paid for under contract pay item 69.
6. Auxiliary Intake Shaft excavation paid for under contract pay item 71.

REFERENCE

A-0D10-2 (Sheet 77)

Explanation of section cutting plane Indicator

B — Section
80 — Sheet where section is shown.

On section the number shown indicates sheet where section is cut.

1" = 20'-0" — 0 5 10 15 20 40 60 80 100 — SCALE OF FEET

STATE OF CALIFORNIA
RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION

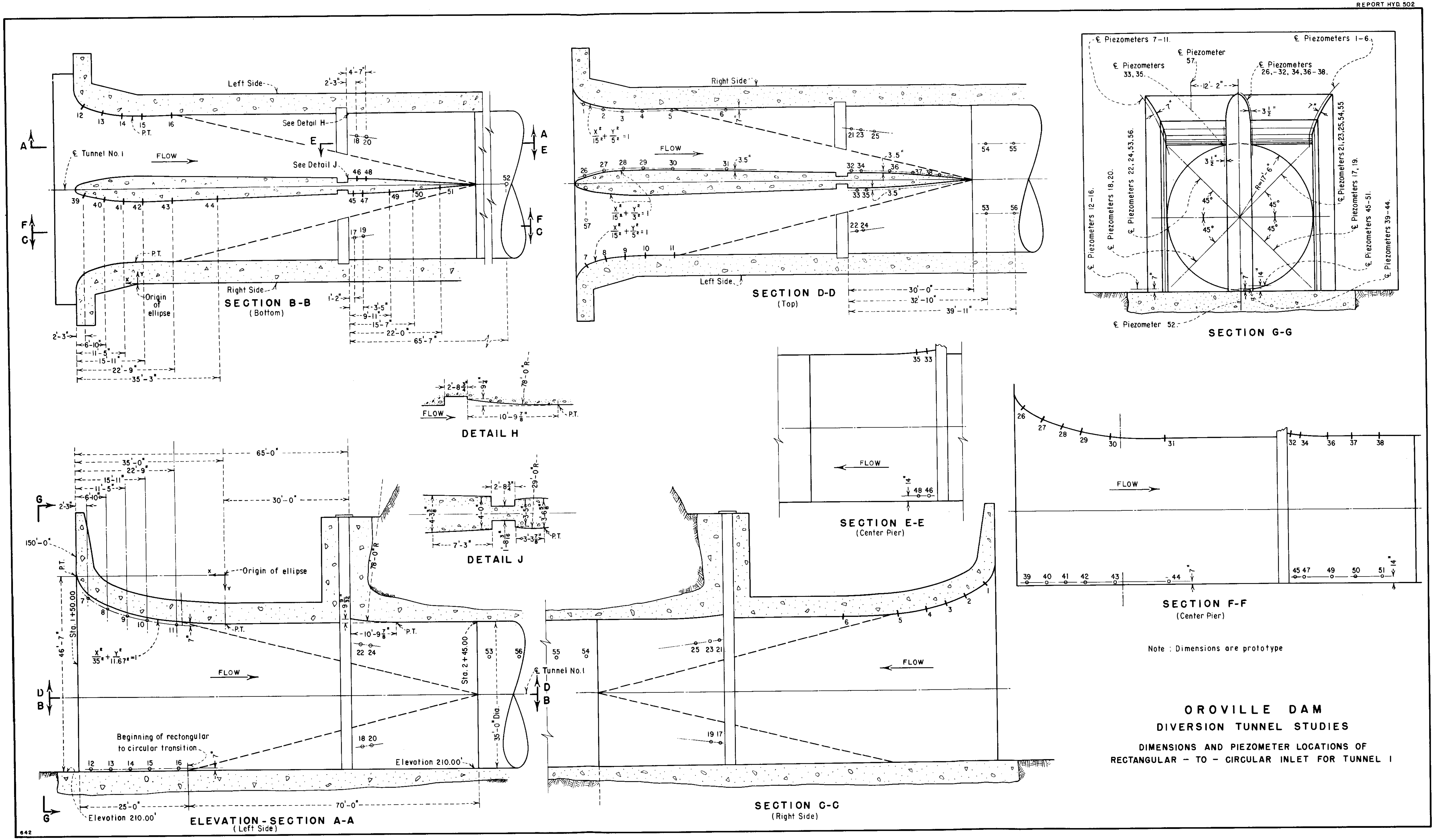
STATE WATER FACILITIES
OROVILLE DIVISION
OROVILLE DAM
DIVERSION TUNNEL NO. 2
INTAKE PORTAL
PLAN

SUBMITTED: [signature] CHIEF, DAM DESIGN SECTION
APPROVAL RECOMMENDED: [signature] CHIEF, DAMS AND PLANTS DESIGN BRANCH
APPROVAL RECOMMENDED: [signature] DEPUTY DIVISION ENGINEER
APPROVED: [signature] DIVISION ENGINEER, REG. C. E. NO. 6900
DATE: MAR 17 1962

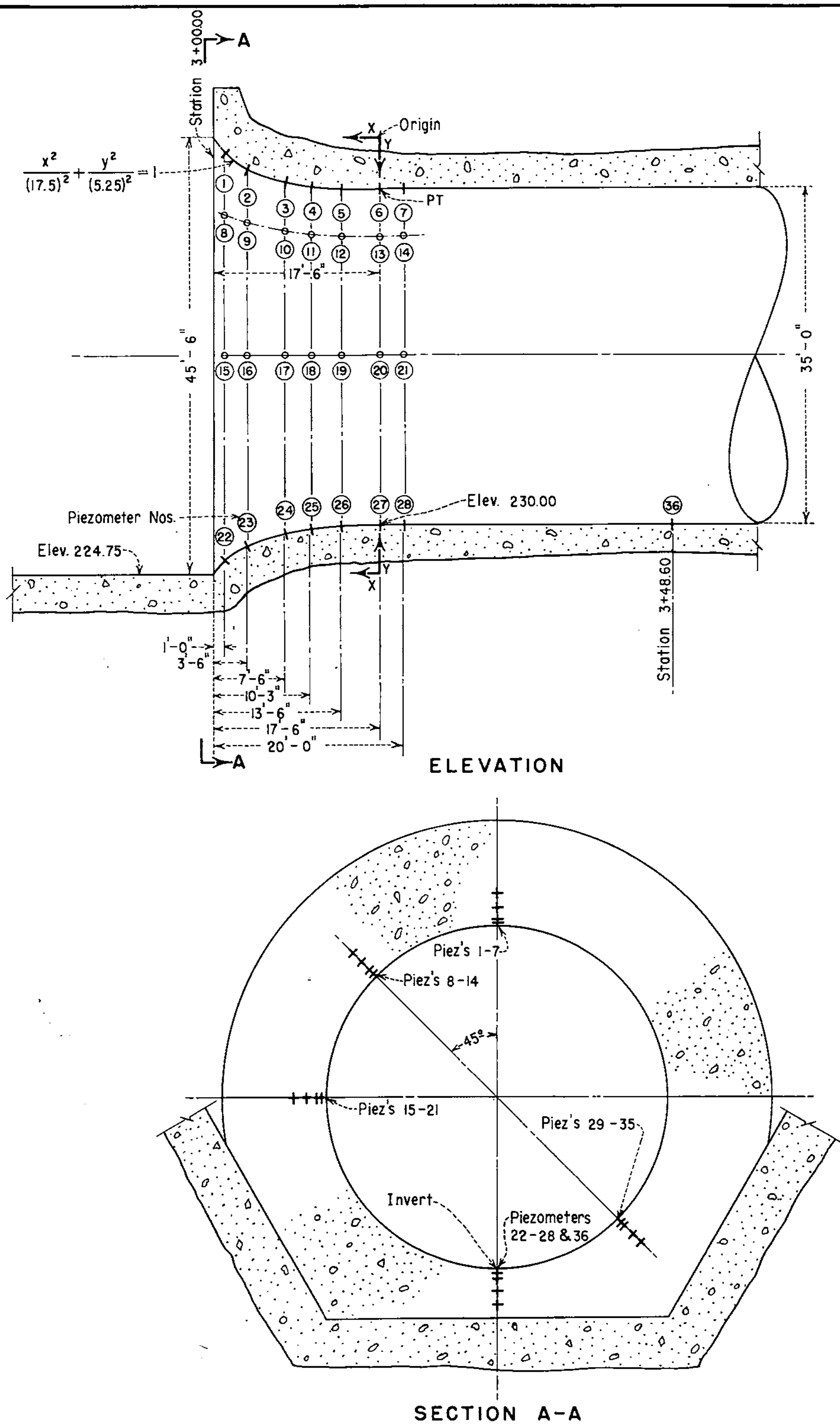
REV.	DATE	DESCRIPTION	SUB.	APP'D

DESIGNED	DATE	DRAWN	DATE	CHECKED	DATE	REVIEWED	DATE
R. Cameron F. Dubar		B. Kotani		C. Lenning		S. Linn G. Dukleth	

DRAWING NO. A-0D10-4
SHEET NO. 79



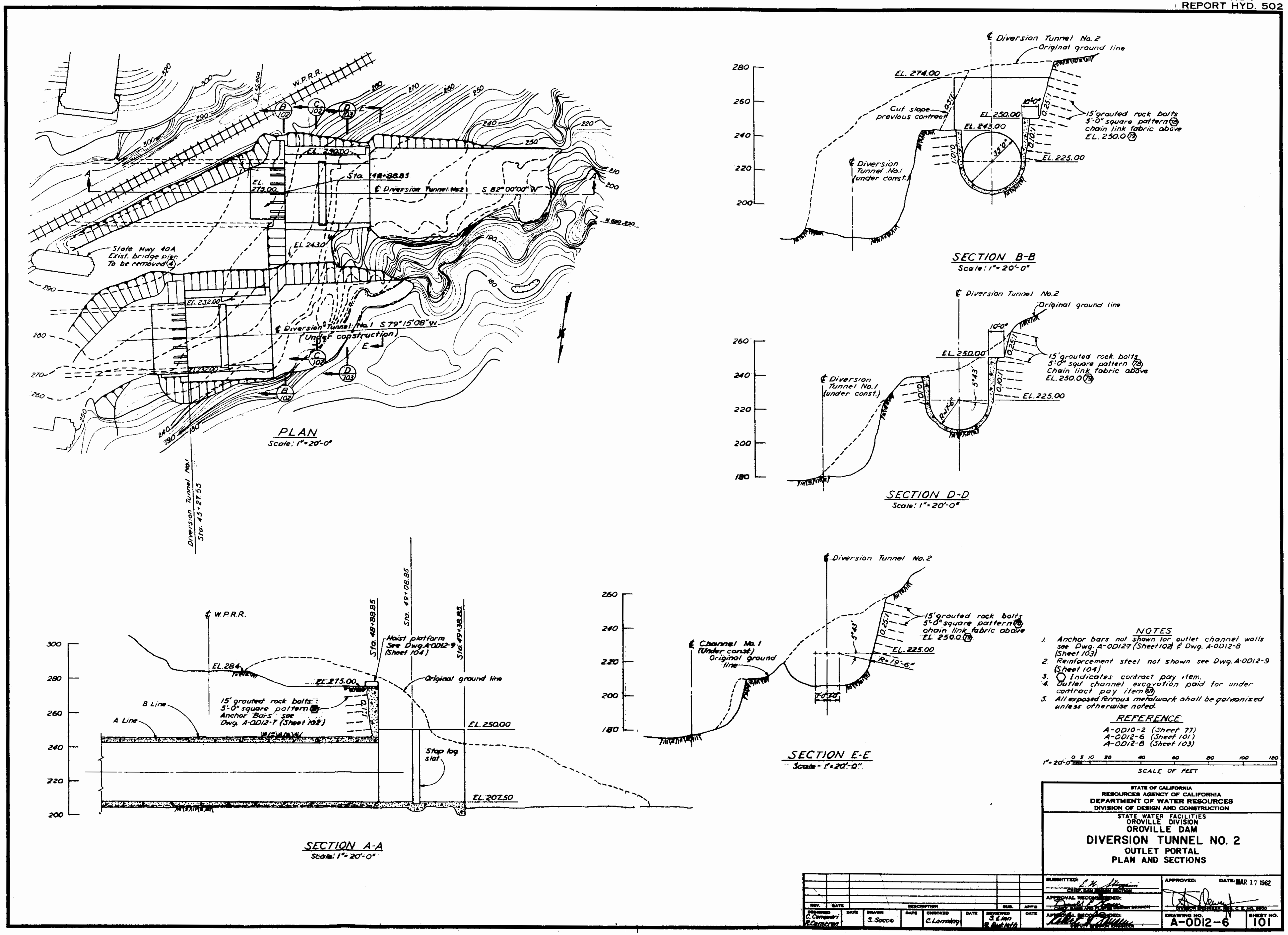
OROVILLE DAM
DIVERSION TUNNEL STUDIES
DIMENSIONS AND PIEZOMETER LOCATIONS OF
RECTANGULAR - TO - CIRCULAR INLET FOR TUNNEL 1

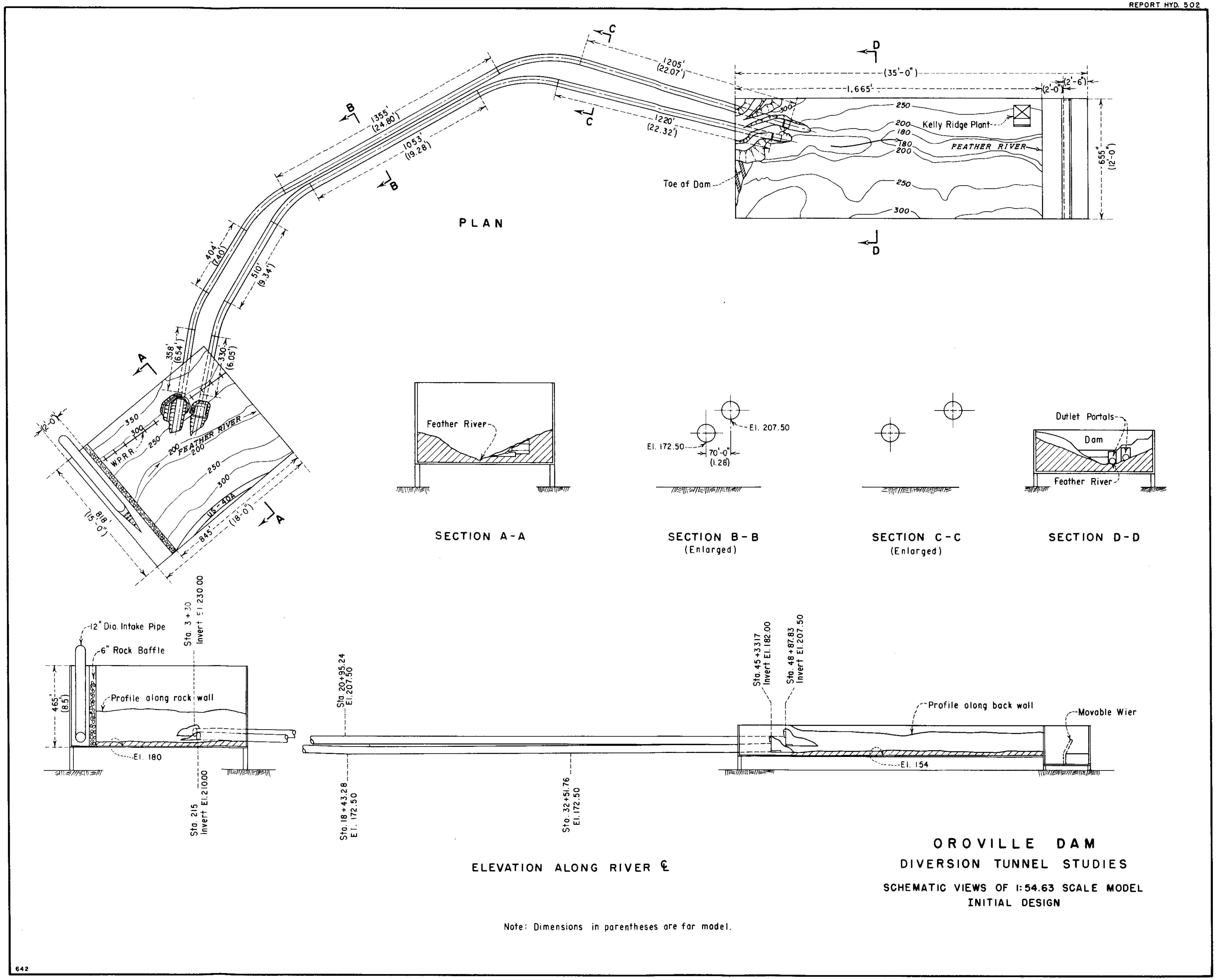


OROVILLE DAM

DIVERSION TUNNEL STUDIES

DIMENSIONS AND PIEZOMETER LOCATIONS FOR CIRCULAR BELLMOUTH INLET FOR TUNNEL 2


PLAN
Scale: 1" = 20'-0"
W.P.R.R.
State Hwy 40A
Exist. bridge pier
To be removed
EL. 250.00
EL. 275.00
Sta. 48+88.85
℄ Diversion Tunnel No. 2 S 82°00'00" W
EL. 243.0
EL. 232.00
℄ Diversion Tunnel No. 1 S 79°15'08" W
(Under construction)
Diversion Tunnel No. 1
Sta. 45+27.55
SECTION B-B
Scale: 1" = 20'-0"
℄ Diversion Tunnel No. 2
Original ground line
EL. 274.00
Cut slope previous contract
EL. 250.00
EL. 243.00
EL. 225.00
15' grouted rock bolts 5'-0" square pattern chain link fabric above EL. 250.0
℄ Diversion Tunnel No. 1 (under const.)
SECTION D-D
Scale: 1" = 20'-0"
SECTION E-E
Scale - 1" = 20'-0"
℄ Channel No. 1 (Under const.)
Original ground line
EL. 225.00
R = 19'-6"
SECTION A-A
Scale: 1" = 20'-0"
℄ W.P.R.R.
EL. 284
EL. 275.00
Hoist platform See Dwg. A-0D12-9 (Sheet 104)
Sta. 48+88.85
Sta. 49+08.85
Sta. 49+38.85
Original ground line
B Line
A Line
15' grouted rock bolts 5'-0" square pattern
Anchor Bars see Dwg. A-0D12-7 (Sheet 102)
EL. 250.00
Stop log slot
EL. 207.50
NOTES
1. Anchor bars not shown for outlet channel walls see Dwg. A-0D12-7 (Sheet 102) & Dwg. A-0D12-8 (Sheet 103)
2. Reinforcement steel not shown see Dwg. A-0D12-9 (Sheet 104)
3. ○ Indicates contract pay item.
4. Outlet channel excavation paid for under contract pay item
5. All exposed ferrous metalwork shall be galvanized unless otherwise noted.
REFERENCE
A-0D10-2 (Sheet 77)
A-0D12-6 (Sheet 101)
A-0D12-8 (Sheet 103)
SCALE OF FEET
STATE OF CALIFORNIA
RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION
STATE WATER FACILITIES
OROVILLE DIVISION
OROVILLE DAM
DIVERSION TUNNEL NO. 2
OUTLET PORTAL
PLAN AND SECTIONS
APPROVED: DATE: MAR 17 1962
DRAWING NO. A-0D12-6
SHEET NO. 101



OROVILLE DAM
DIVERSION TUNNEL STUDIES
SCHEMATIC VIEWS OF 1:54.63 SCALE MODEL
INITIAL DESIGN

A. Wooden contour templates for downstream topography.

B. Concrete for final topography was applied over expanded metal lath that conformed to contours. Note approach channels and tunnel inlet structures.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

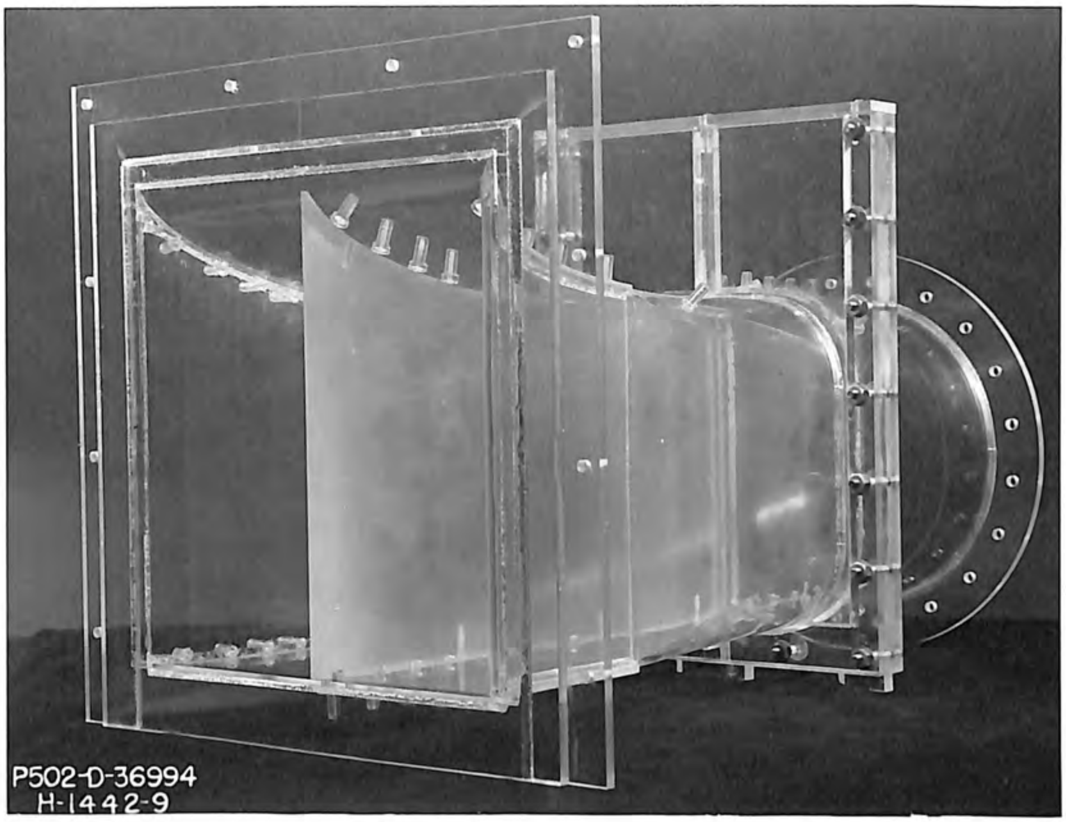
Construction of 1:54.63 Model

A. 7.69-inch plastic pipes represented the 35-foot-diameter tunnels and extended from head box (rear) to tail box (left foreground).

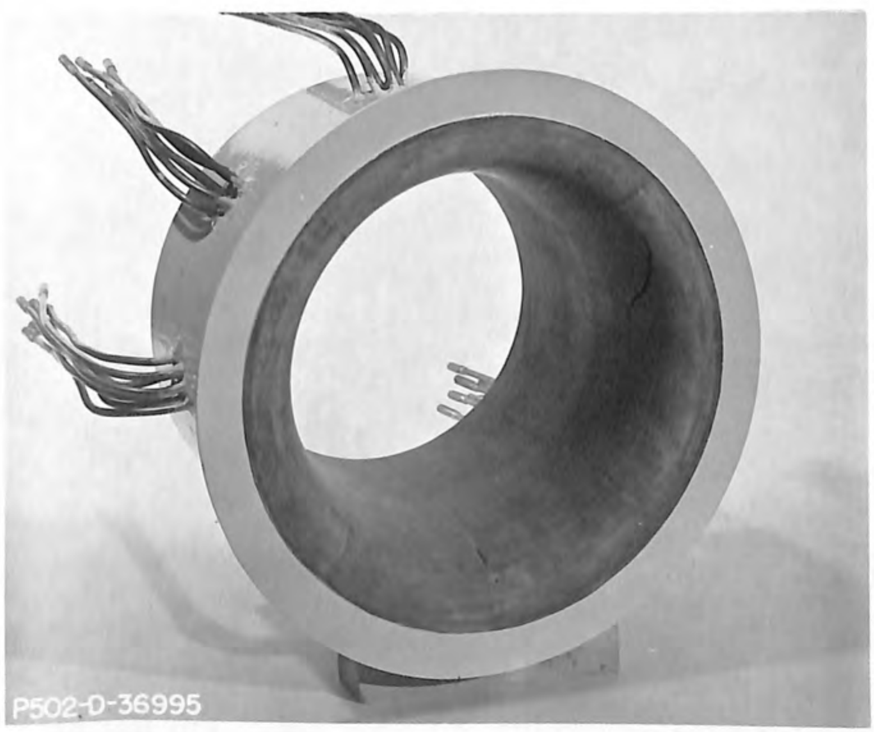
B. The tunnels discharge into the river channel represented in the tail box.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

The Completed 1:54.63 Model



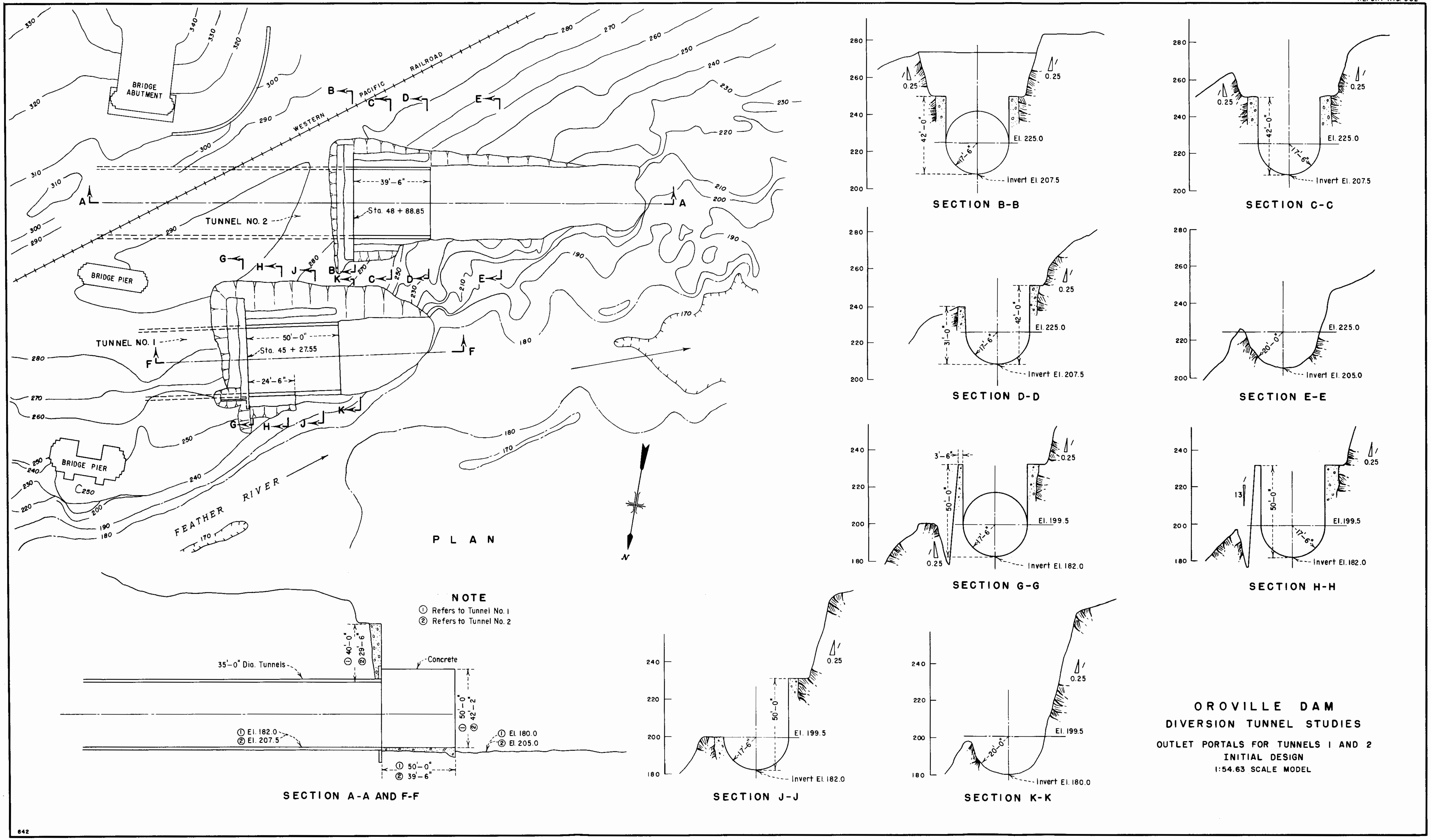
A. Rectangular-to-circular inlet for Tunnel 1.



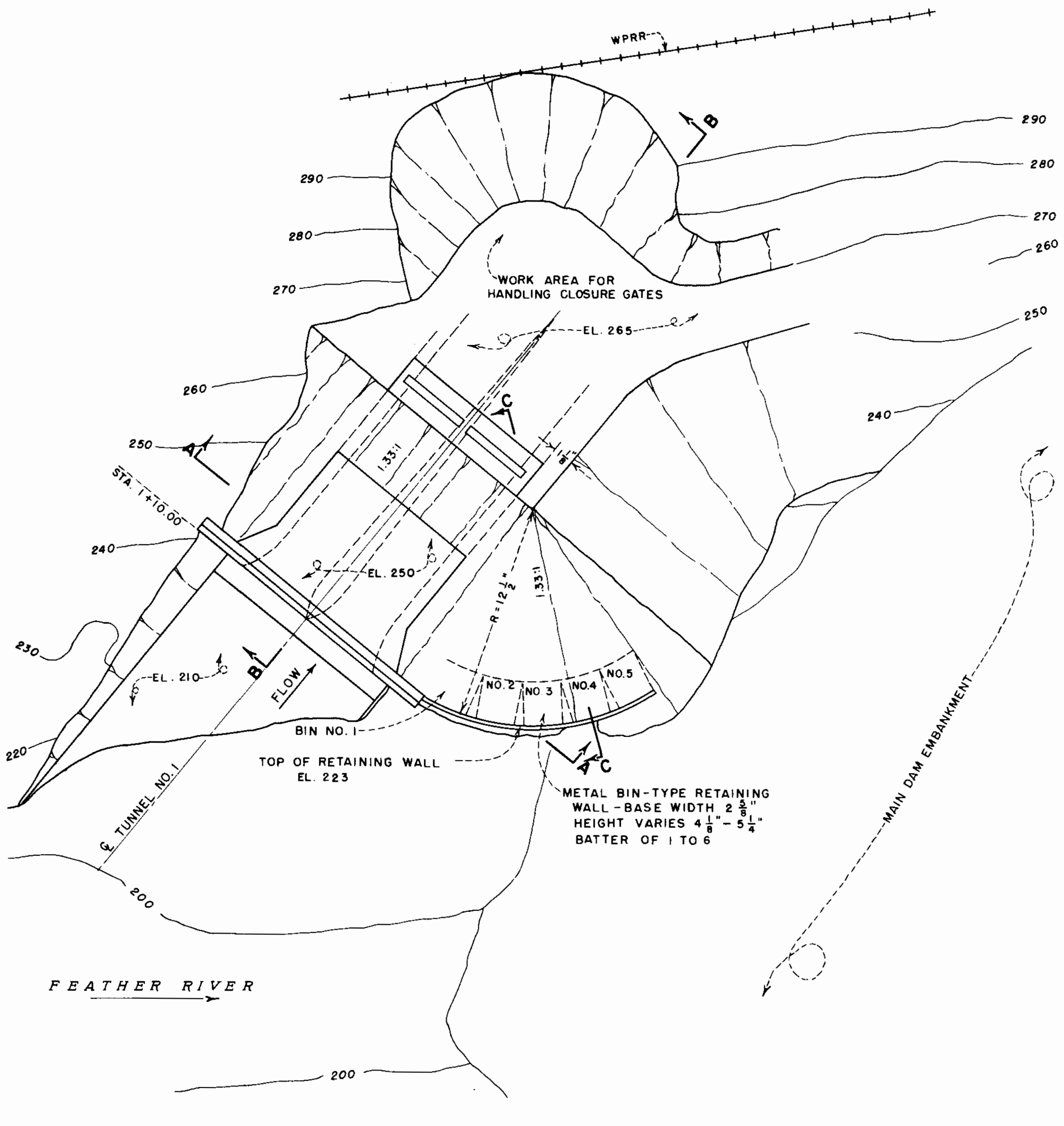
B. Circular inlet for Tunnel 2.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

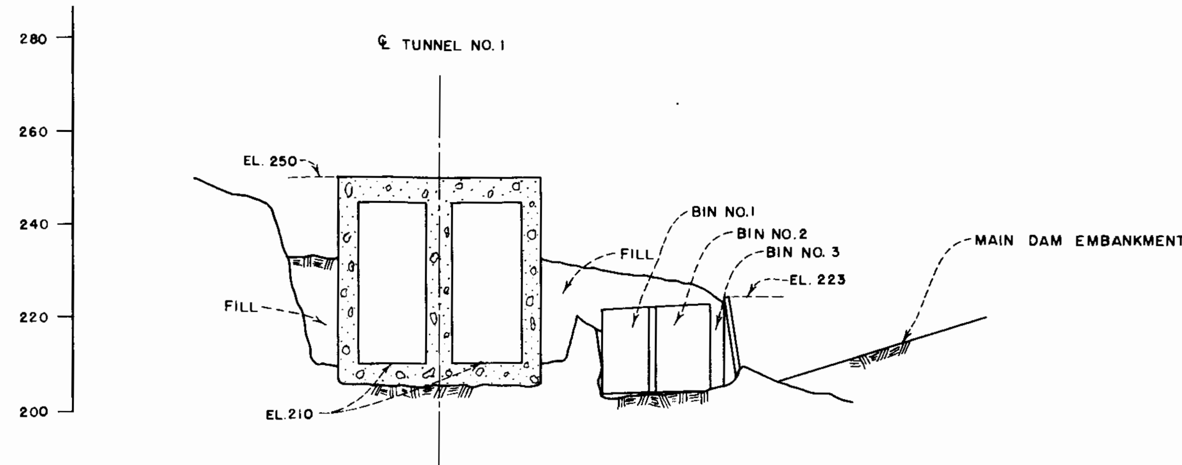
Bellmouth Inlets for 1:54.63 Scale Model



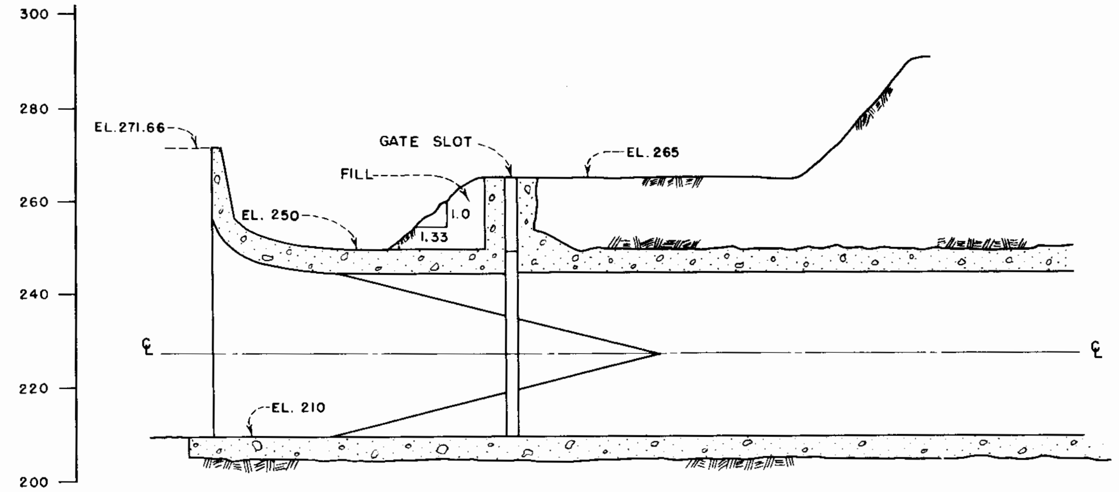
OROVILLE DAM
DIVERSION TUNNEL STUDIES
OUTLET PORTALS FOR TUNNELS 1 AND 2
INITIAL DESIGN
1:54.63 SCALE MODEL



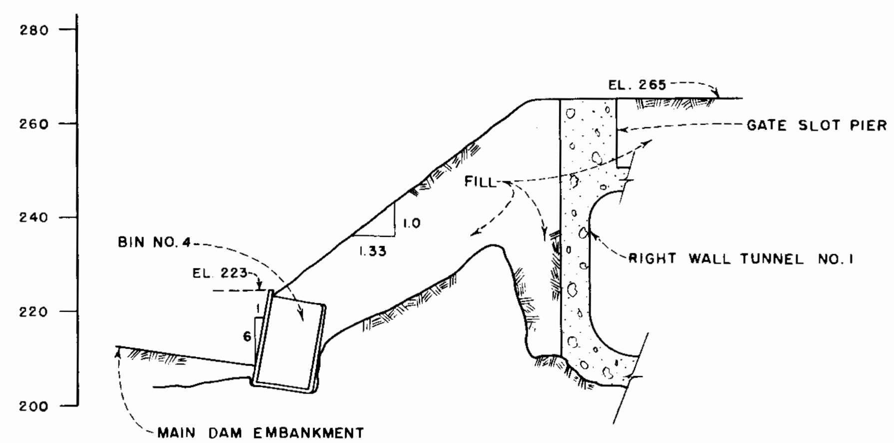
P L A N



SECTION A-A



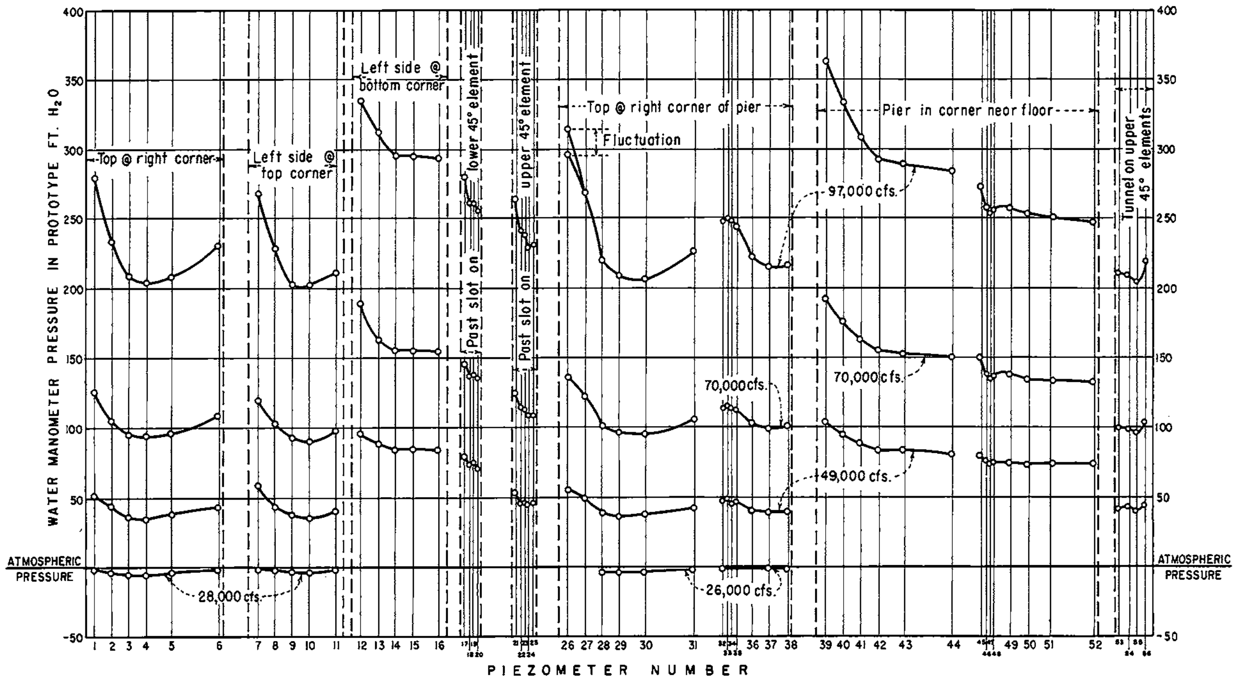
SECTION B-B



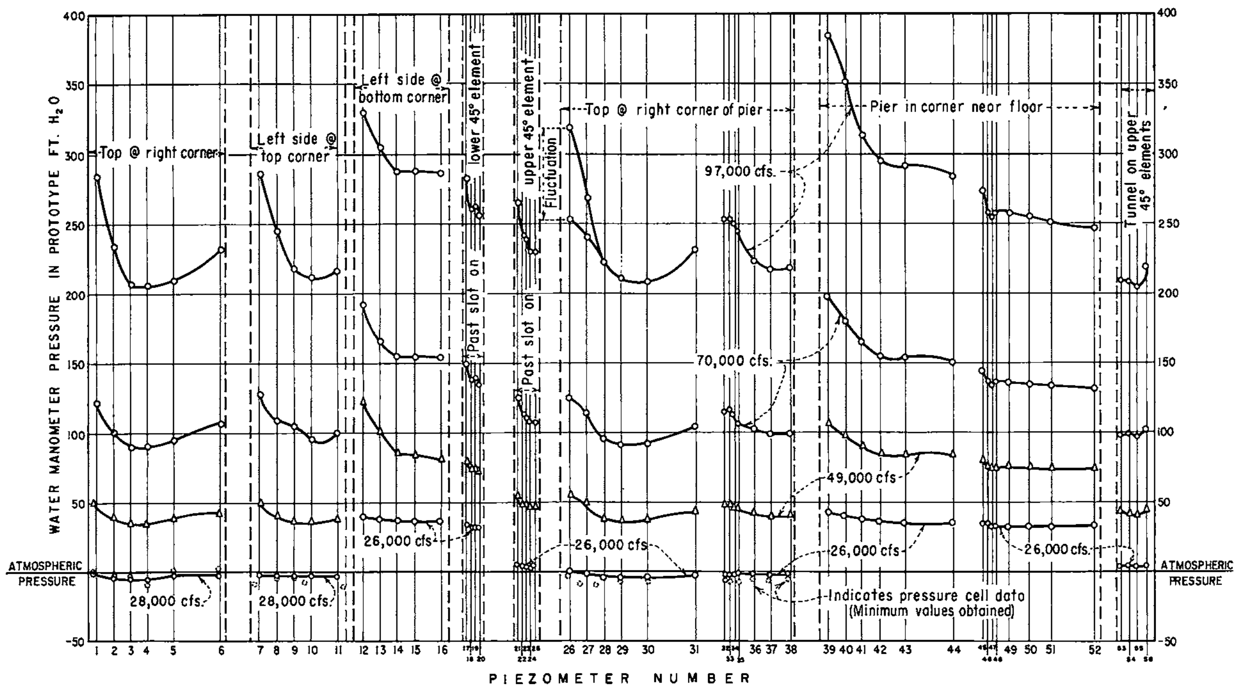
SECTION C-C

OROVILLE DAM
DIVERSION TUNNEL STUDIES
RELOCATED INLET PORTAL - TUNNEL 1
1:54.63 SCALE MODEL

642



A. TUNNEL 1 OPERATING ONLY



B. BOTH TUNNELS OPERATING

Piezometer locations shown in Figure 5

OROVILLE DAM

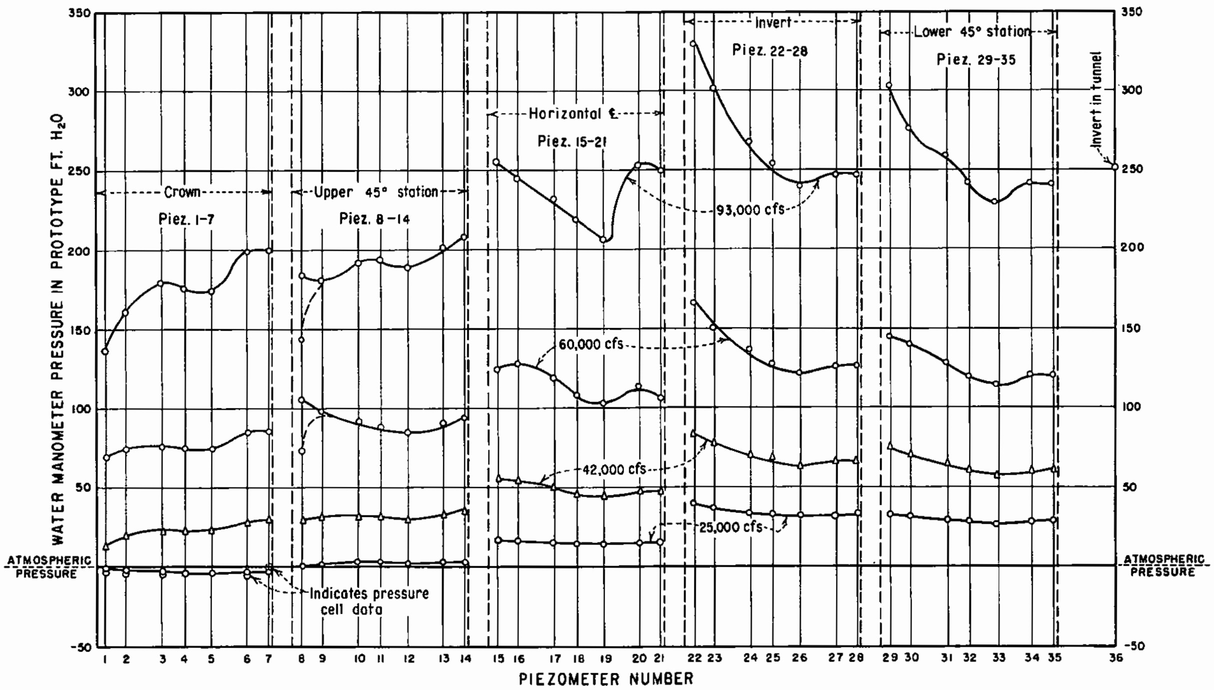
DIVERSION TUNNEL STUDIES

BELLMOUTH INLET PRESSURES

TUNNEL 1

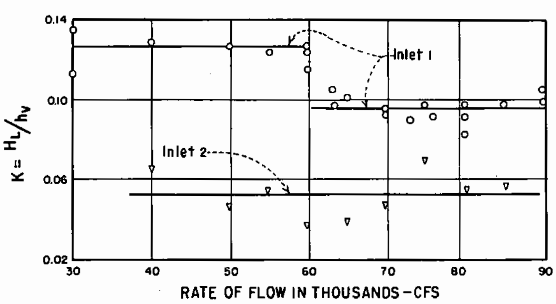
DATA FROM 1:54.63 SCALE MODEL

642



Piezometer Locations Shown in Figs 5 and 6

A. PRESSURES – BOTH TUNNELS OPERATING



H_L = Res. El. minus Total Head measured 9'–11" downstream from transition in Tunnel 1, and 48'–7" from transition in Tunnel 2.
h_v = Av. velocity head in tunnel ($V^2/2g$)

B. LOSS FACTORS – SINGLE TUNNEL OPERATION

OROVILLE DAM
DIVERSION TUNNEL STUDIES

BELLMOUTH INLET PRESSURES – TUNNEL 2, AND LOSS FACTORS – INLETS 1 AND 2

DATA FROM 1:54.63 SCALE MODEL

Figure 16
Report Hyd-502

A. Time exposure showing small vortex at Inlet 1 (left foreground) and large vortex at Inlet 2.

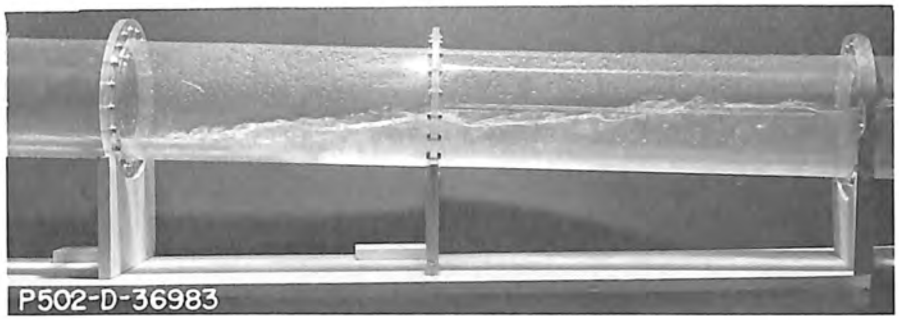
B. 30-foot tree being drawn into vortex at Inlet 2.

C. The vortices carry air into the tunnels.

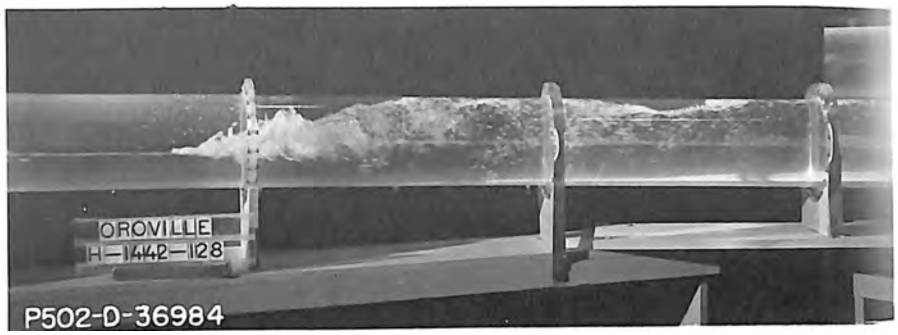
Inlet # 2 ℄ = El 2??.5
Inlet # 1 ℄ = El 227.5

OROVILLE DAM
DIVERSION TUNNEL STUDIES

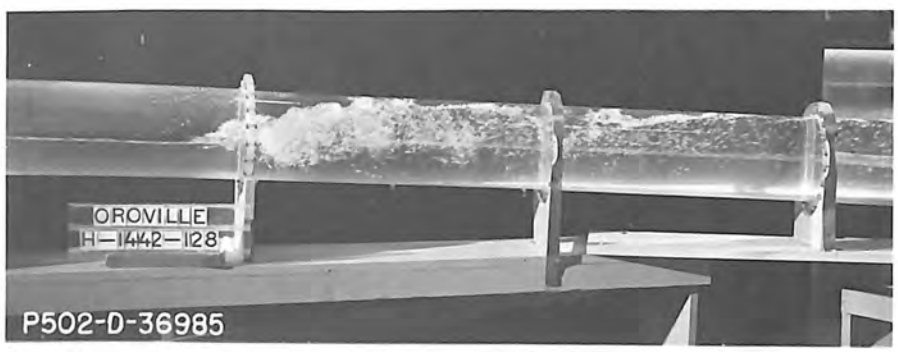
Vortices at Inlet Structures
Q Total = 100,000 cfs Res. El. = 348
1:54.63 Scale Model



A. Q = 4,000, Station 27+57--Stable jump.



B. Q = 16,000, Station 27+57--A large bubble accumulates just past the jump and then moves upstream to vent at the jump.

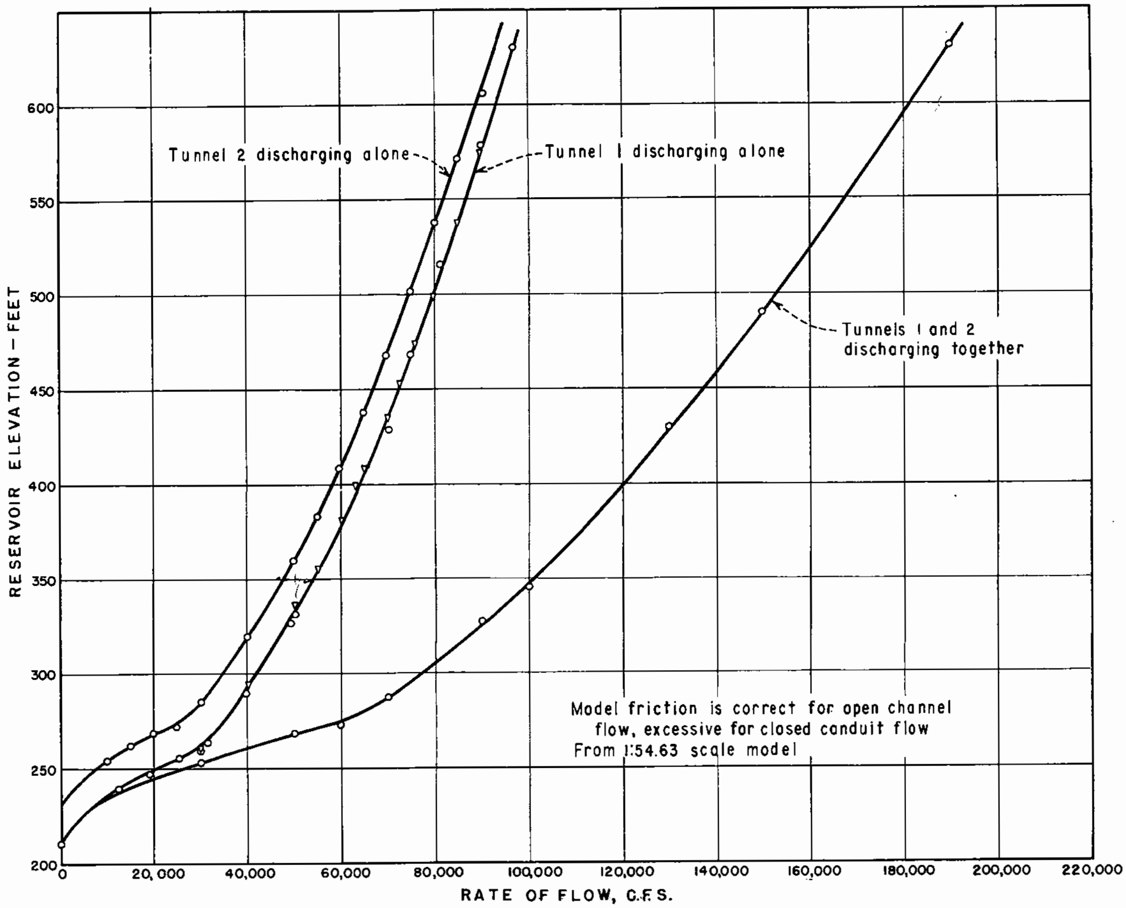


C. Q = 16,000, Station 27+57--The bubble venting.

D. Q = 16,000. A second, more stable bubble frequently occurs at Station 30+00, with air entering at the upstream end and leaving at the downstream end.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

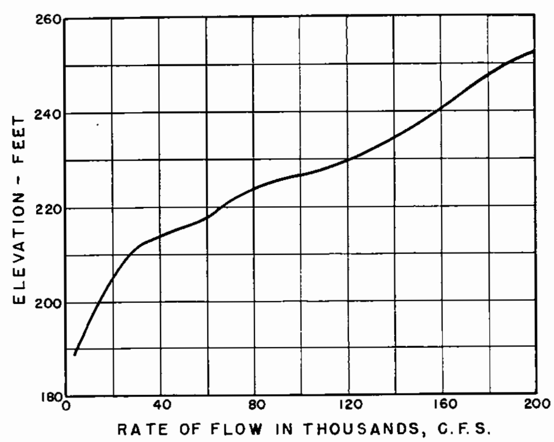
Flow Conditions in Tunnel 1 at Low Discharges
1:54.63 Scale Model



A. TUNNEL FLOW VS RESERVIOR ELEVATION

Inlet " 2 217.5 ℄ elev

Inlet # 1 = 227.5



B. TAILWATER CURVE

OROVILLE DAM

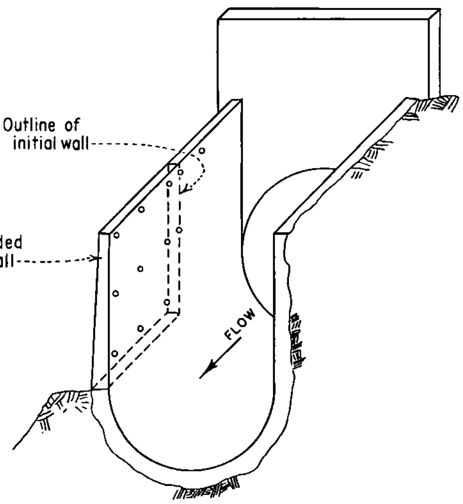
DIVERSION TUNNEL STUDIES

TUNNEL DISCHARGE CAPACITY AND TAILWATER CURVES

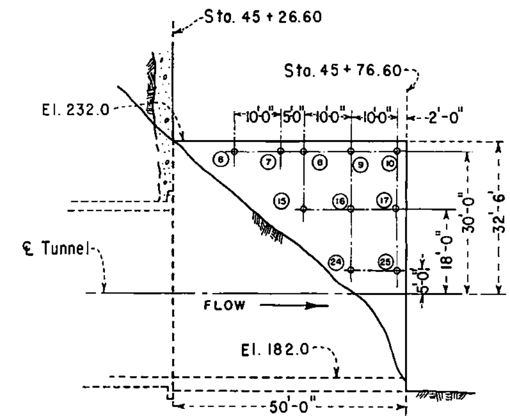
Clockwise eddy in river channel pushed jet from Tunnel 1 against rock underlying Portal 2.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

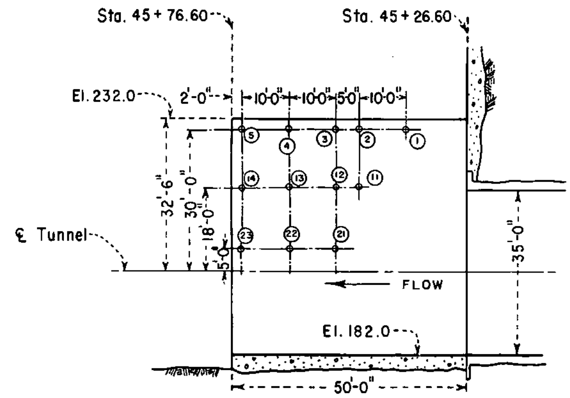
Flow Conditions in River Channel
Q = 50,000 cfs, Initial Outlet Portals
1:54.63 Scale Model



OUTLET PORTAL
FROM HILLSIDE DOWNSTREAM



OUTSIDE OF GUIDE WALL



INSIDE OF GUIDE WALL

NOTES

Values in Pressure Table are in feet of water prototype.
Pressures on the outside of the portal wall were somewhat higher with the deflector wall removed.
Tailwater changes directly affect the portal wall pressures.
* Indicates short duration pips
Numbers circled are pizometer numbers.

PIEZOMETER NUMBER	WATER MANOMETER		PRESSURE CELL			REMARKS
	FLOW CONDITIONS	AVERAGE PRESSURE	MINIMUM PRESSURE	MAXIMUM PRESSURE	AVERAGE PRESSURE	
1 THRU 10						Above Ave. W.S. for all conditions
11	Q=190,000 cfs - wall 2 Spoil pos. 1 - T.W. = 250					Piez. out of water most of the time
12			7	11	9	Piez. out of water most of the time
13						Piez. out of water most of the time
14			-1	20	6	Piez. out of water some of the time
15		4	-2	6	3	
16		4	-4	3	0	
17		4	-7	1	-2	
21		10				
22		10				
23		13				
24		18				
25		17				
11	Q=190,000 cfs - wall 2 Spoil pos. 1 - T.W. = 255					Piez. out of water most of the time
12			0	21	9	Piez. out of water most of the time
13						Piez. out of water most of the time
14			*-14	26	5	Piez. occasionally out of the water
15		12	7	15	9	
16		11	4	10	7	
17		9	*-10	9	4	
21		12				
22		14				
23		20				
24		25				
25		24				

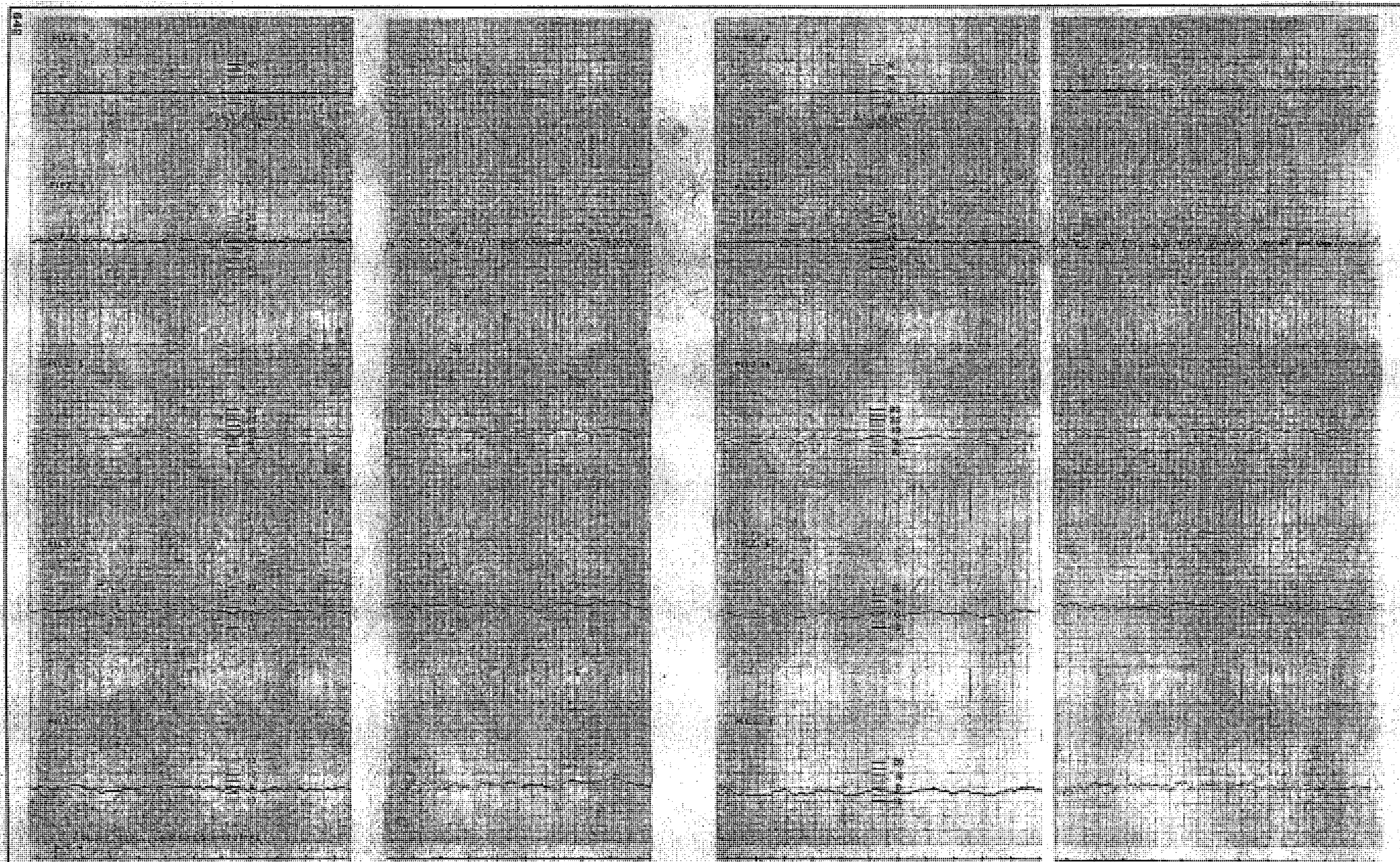
OROVILLE DAM

DIVERSION TUNNEL STUDIES

PIEZOMETER LOCATIONS AND PRESSURES IN RIGHT GUIDE WALL OF OUTLET PORTAL NO. 1

FROM 1:54.63 SCALE MODEL

642



ELEV. 250 TAILWATER — ELEV. 255 TAILWATER
WITHOUT DEFLECTOR WALL

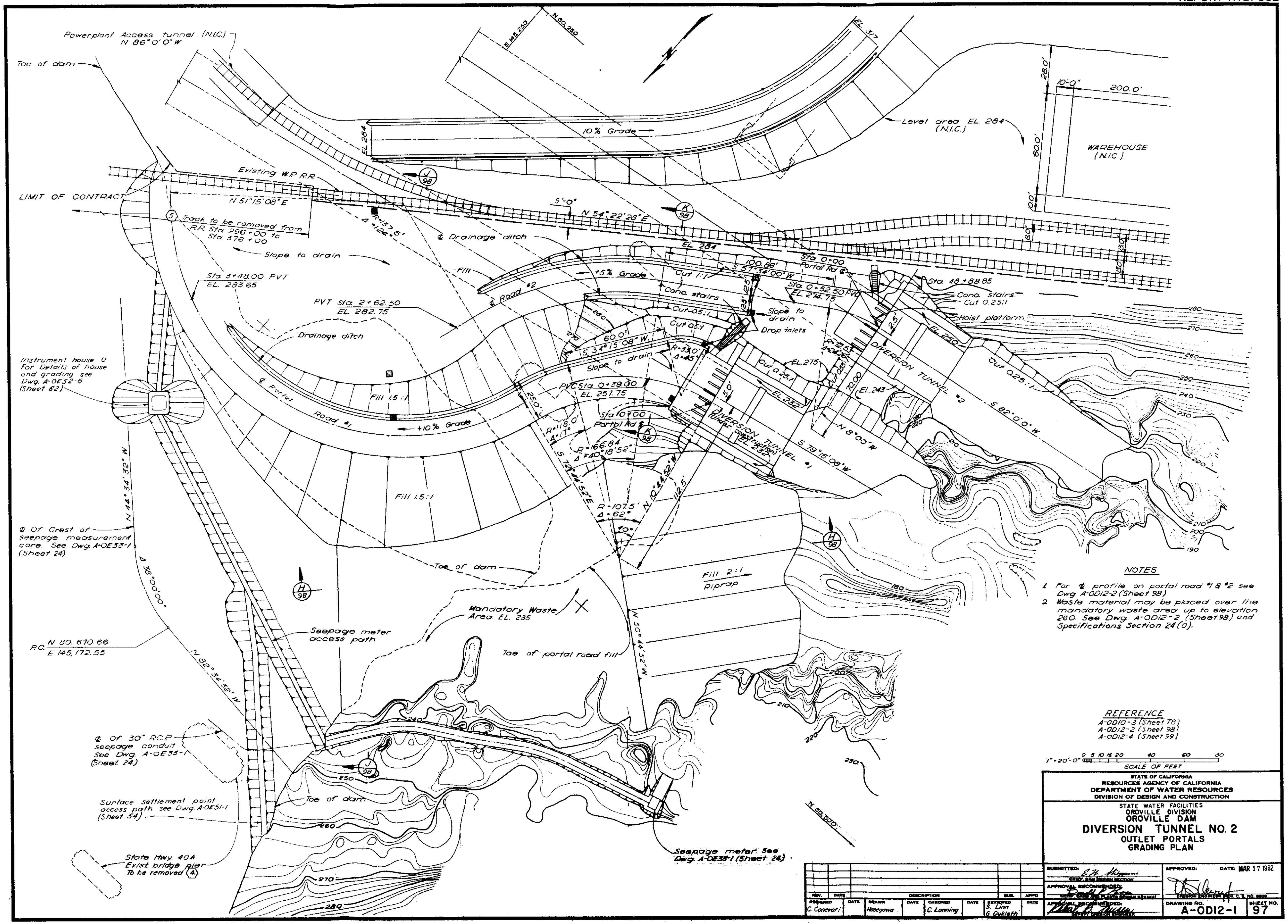
ELEV. 250 TAILWATER — ELEV. 255 TAILWATER
WITH DEFLECTOR WALL NO. 2

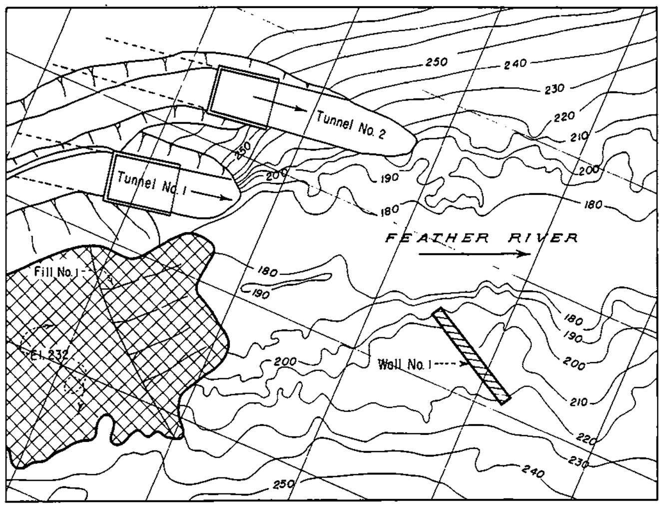
Q = 190,000 CFS, FILL NO. 1 IN PLACE

OROVILLE DAM DIVERSION TUNNEL STUDIES

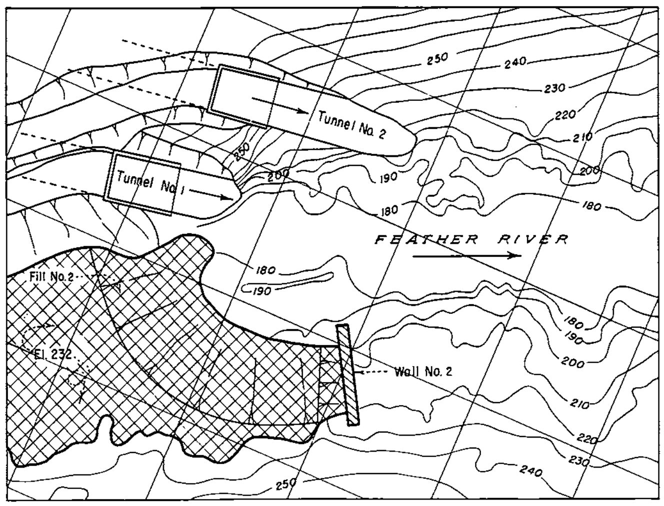
PRESSURES ON RIGHT GUIDE WALL OF OUTLET PORTAL 1

Data from 1:54.63 scale model

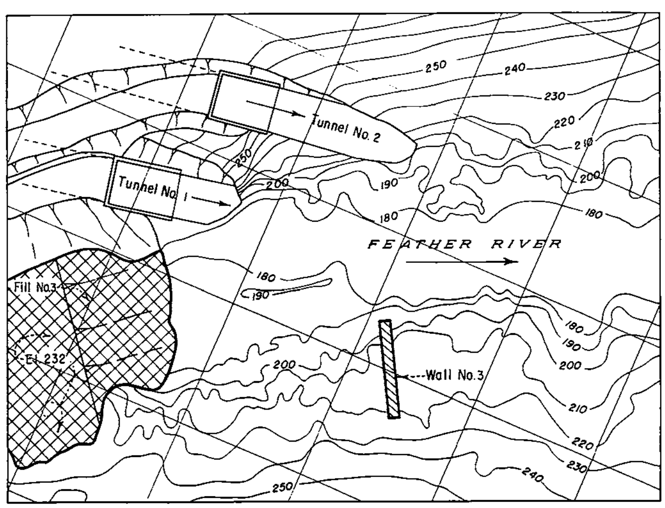

Powerplant Access tunnel (N.I.C.) N 86°0'0" W
Toe of dam
LIMIT OF CONTRACT
Existing W.P. R.R.
N 51°15'08" E
Track to be removed from R.R. Sta. 296+00 to Sta. 576+00
Slope to drain
Drainage ditch
10% Grade
Level area EL 284 (N.I.C.)
WAREHOUSE (N.I.C.)
200.0'
N 54°22'28" E
Sta 3+48.00 PVT EL. 283.65
PVT Sta. 2+62.50 EL. 282.75
Road #2
+5% Grade
Cut 1:1
Conc. stairs
Sta 0+00 Portal Rd
Sta 0+52.50 PVT EL 274.75
Sta 48+88.85
Conc. stairs Cut 0.25:1
Hoist platform
Slope to drain
Drop inlets
Instrument house U For Details of house and grading see Dwg. A-OE52-6 (Sheet 62)
Portal Road #1
Fill 1.5:1
+10% Grade
S. 34°15'08" W.
PVC Sta 0+39.00 EL 257.75
Sta 0+00 Portal Rd
DIVERSION TUNNEL #1 (Under construction)
S. 79°15'08" W
DIVERSION TUNNEL #2
S 82°0'0" W
N 8°00' W
Cut 0.25:1
Fill 1.5:1
Fill 2:1 Riprap
Toe of dam
Mandatory Waste Area EL 235
Toe of portal road fill
N 50°44'52" W
Of Crest of seepage measurement core. See Dwg. A-OE33-1 (Sheet 24)
N 48°34'32" W
R.C. N 80,670.66 E 145,172.55
N 82°34'32" W
Seepage meter access path
Of 30" R.C.P. seepage conduit See Dwg. A-OE33-1 (Sheet 24)
Surface settlement point access path see Dwg. A-OE51-1 (Sheet 54)
State Hwy. 40A Exist. bridge pier To be removed
Seepage meter. See Dwg. A-OE33-1 (Sheet 24)
N 80,500
NOTES
1. For ℄ profile on portal road #1 & #2 see Dwg. A-OD12-2 (Sheet 98)
2. Waste material may be placed over the mandatory waste area up to elevation 260. See Dwg. A-OD12-2 (Sheet 98) and Specifications Section 24(0).
REFERENCE
A-OD10-3 (Sheet 78)
A-OD12-2 (Sheet 98)
A-OD12-4 (Sheet 99)
1" = 20'-0"
SCALE OF FEET
STATE OF CALIFORNIA
RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION
STATE WATER FACILITIES
OROVILLE DIVISION
OROVILLE DAM
DIVERSION TUNNEL NO. 2
OUTLET PORTALS
GRADING PLAN
DATE: MAR 17 1962
DESIGNED C. Conevari
DRAWN Hasegawa
CHECKED C. Lanning
REVIEWED S. Linn G. Dukleth
DRAWING NO. A-OD12-1
SHEET NO. 97



A. FILL NO. 1 - SPUR WALL NO. 1



B. FILL NO. 2 SPUR WALL NO. 2



C. FILL NO. 3 - SPUR WALL NO. 3

OROVILLE DAM

DIVERSION TUNNEL STUDIES

RIVER CHANNEL FILLS AND SPUR WALL PLACEMENTS

1:54.63 SCALE MODEL

No Wall

Spur Wall No. 2

Q = 50, 000 cfs, TW = 215. 5

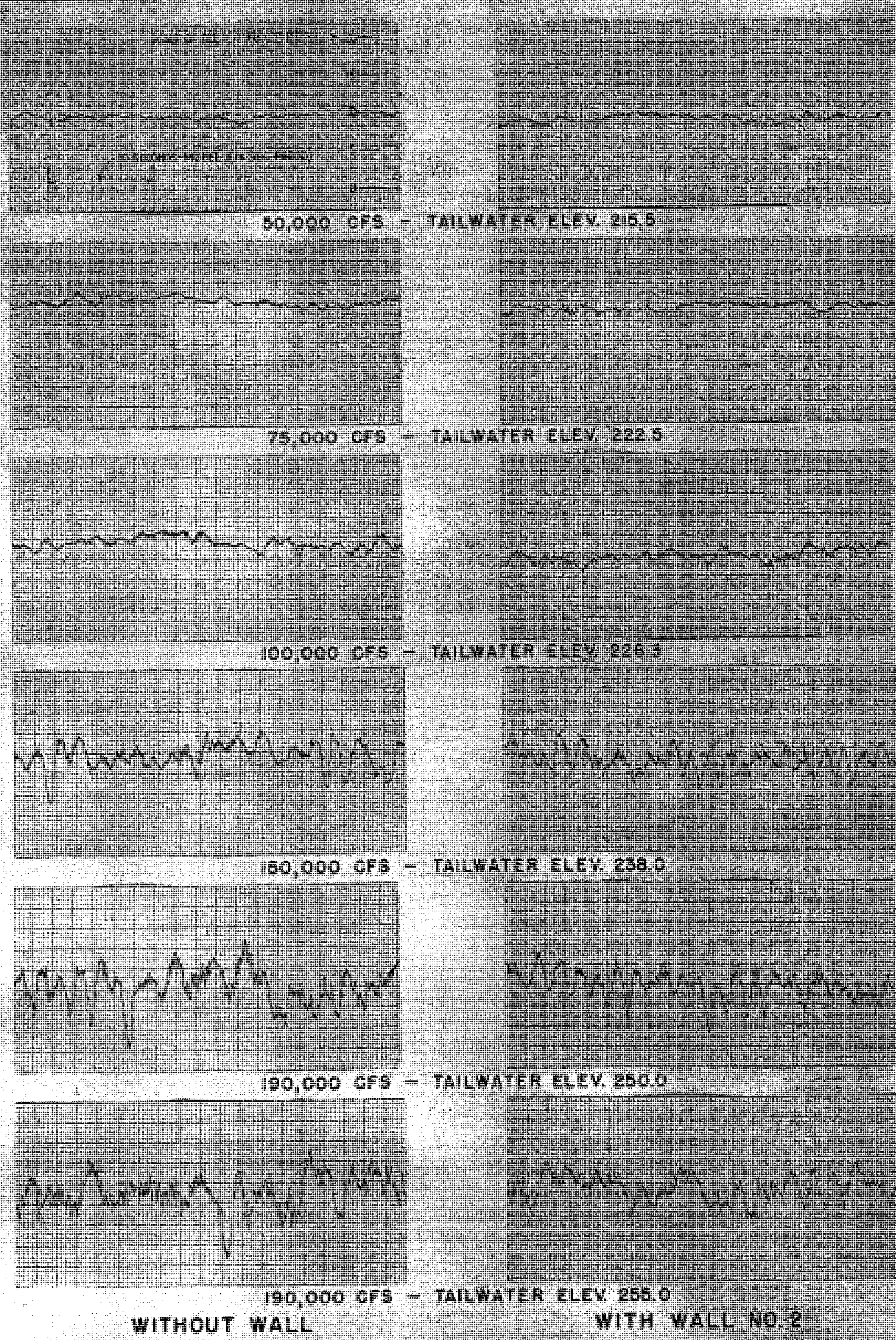
No Wall

Spur Wall No. 2

Q = 150, 000 cfs, TW = 238. 0

OROVILLE DAM DIVERSION TUNNEL STUDIES

Flow With and Without Spur Wall No. 2--Fill No. 1
1:54. 63 Scale Model



Waves measured 45 feet to right and 10 feet upstream from end of right guide wall on Tunnel 1. No fills.

OROVILLE DAM DIVERSION TUNNEL STUDIES

WAVES AT OUTLET PORTAL 1 WITH AND WITHOUT SPUR WALL NO. 2

Data from 1:54.63 scale model

A. No fill in channel, but with Spur Wall No. 2.
Q = 100,000 cfs, TW = 226.0.

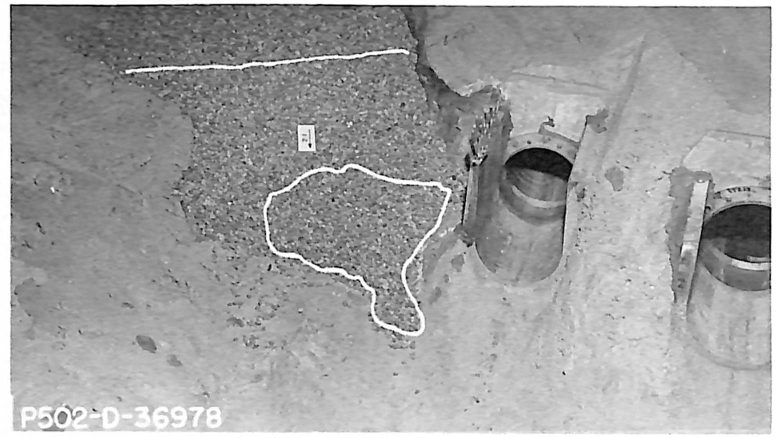
B. No fill, but with Spur Wall No. 2.
Q = 190,000, TW = 250.0. 15-foot surges occur at toe of dam.

C. Channel fill protected by vertical wall.
Q = 16,000, TW = 201.0.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

River Channel from Toe of Dam to Portals--With and Without Fill
1:54.63 Scale Model

A. Initial condition representing 13- to 21-inch gravel.

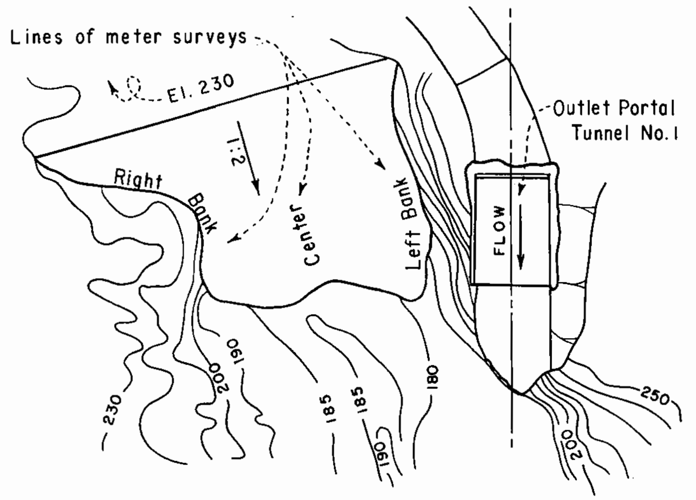


B. 13- to 21-inch gravel after 15 minutes model operation--single tunnel flow--Q = 81,500 cfs.

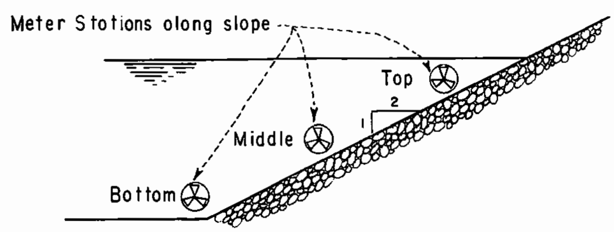
C. Initial condition of 21- to 41-inch riprap blanket 9 feet thick. Arrow shows path of high velocity, shallow flow with single tunnel operation.

D. 21- to 41-inch riprap after 15 minutes model operation--single tunnel flow--Q = 90,000 cfs.

OROVILLE DAM DIVERSION TUNNEL STUDIES



PLAN



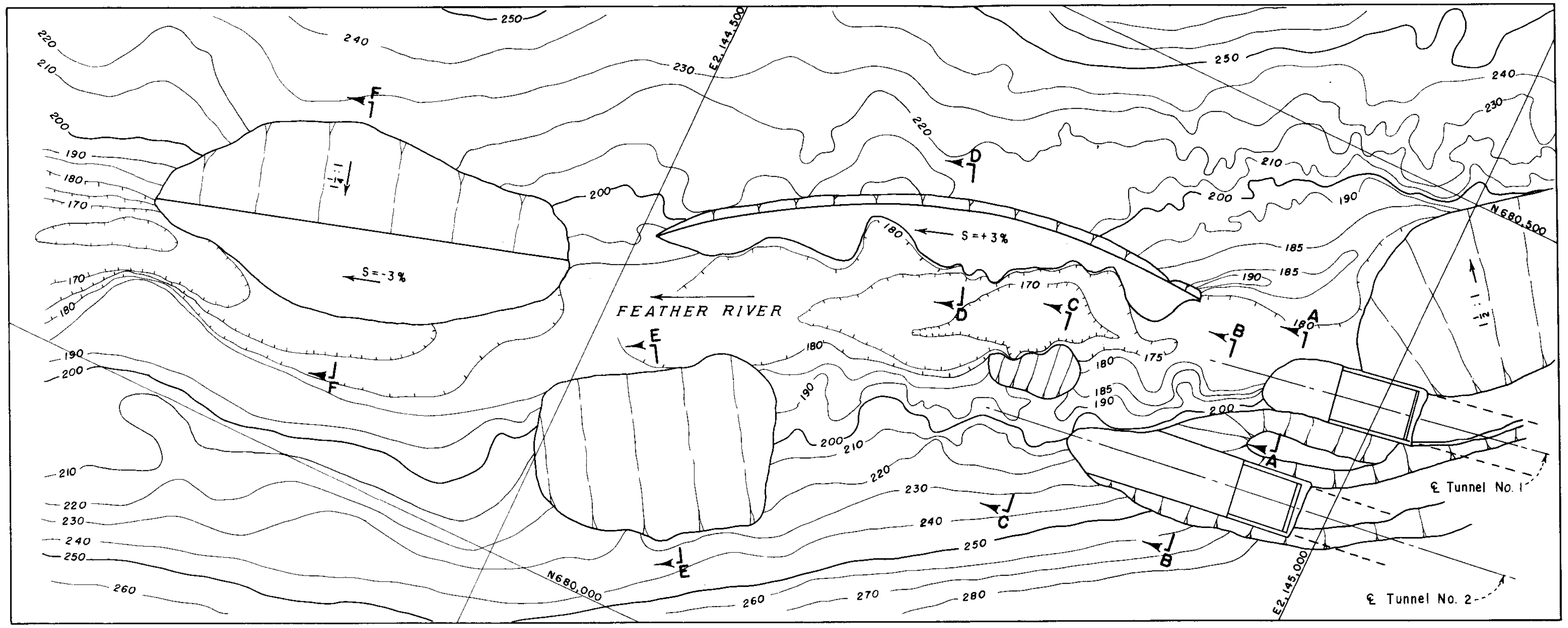
ELEVATION

OPERATION	DISCHARGE (C.F.S.)	RIGHT BANK						CENTER						LEFT BANK					
		BOTTOM		MIDDLE		TOP		BOTTOM		MIDDLE		TOP		BOTTOM		MIDDLE		TOP	
		MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.	MAX.	AVE.
SINGLE TUNNEL	50,000	2.8	2.7	3.8	3.3	5.3	4.2	3.5	3.0	4.1	4.0	3.2	2.8	2.9	2.7	2.3	2.0	2.0	1.5
	81,500	*						13.4	11.7					9.6	8.7				
TWO TUNNEL	90,000	3.6	3.4	4.2	3.5	3.4	3.1	2.8	2.6	1.9	1.8	2.3	1.9	4.0	3.4	3.5	3.4	3.0	2.8
	135,000	3.8	3.3	4.0	3.6	4.9	4.0	4.1	3.1	*				3.5	3.1			4.0	3.8

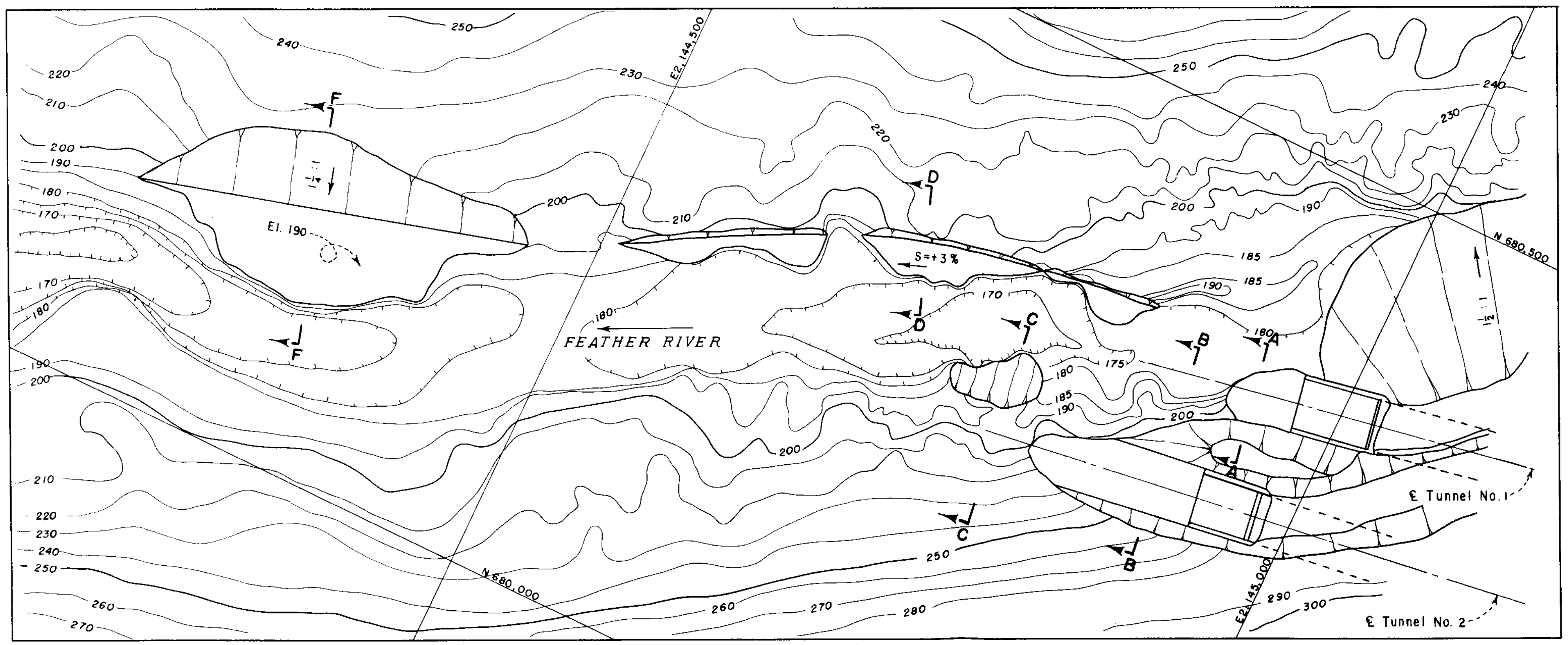
* (Blanks) Velocity reading unobtainable because of irregular flow or shallow water depth

Velocities measured with propeller meter totalizing over 10-second model periods.
Average velocity - Average of several 10-second model periods.
Maximum velocity - highest of the 10-second model integrations.

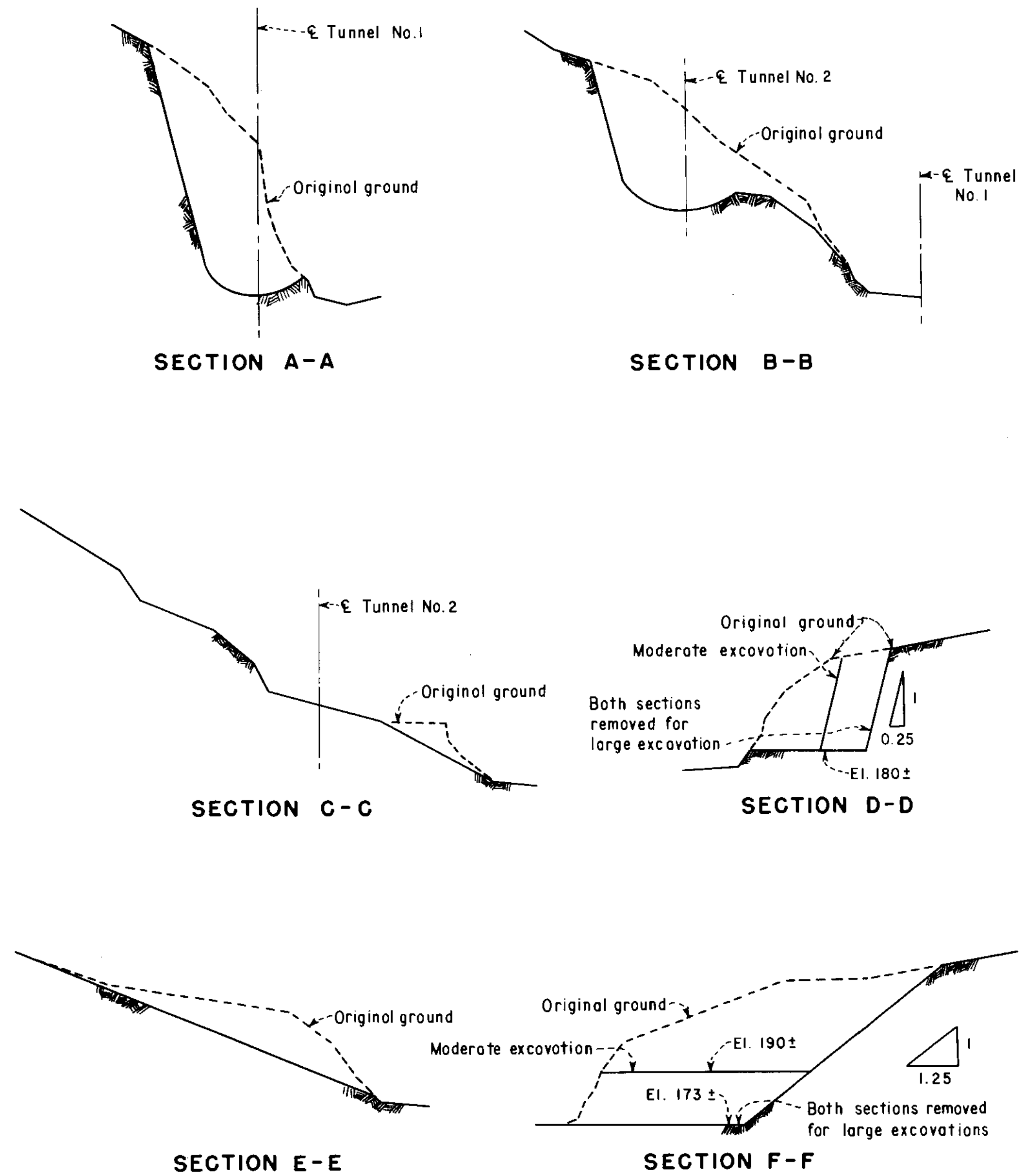
OROVILLE DAM
DIVERSION TUNNELS STUDIES
PROTOTYPE FLOW VELOCITIES SWEEPING ALONG FACE OF FILL NO. 1
DATA FROM 1:54.63 SCALE MODEL



A. LARGE EXCAVATION – BOTH RIVER BANKS



B. MODERATE EXCAVATION – RIGHT RIVER BANK



OROVILLE DAM
DIVERSION TUNNEL STUDIES
EXCAVATIONS OF ROCK OUTCROPS TO STRAIGHTEN RIVER CHANNEL
1:54.63 SCALE MODEL

642

A. Q = 25,000 cfs, TW = 208.5.

B. Q = 50,000 cfs, TW = 217.5.

C. Q = 75,000 cfs, TW = 225.5.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

Flow Conditions with Recommended Design--Low Flows
1:54.63 Scale Model

Figure 31
Report Hyd-502

A. Q = 100,000 cfs, TW = 226.3.

B. Q = 150,000 cfs, TW = 238.0.

C. Q = 190,000, TW = 250.0.

OROVILLE DAM
DIVERSION TUNNEL STUDIES

Flow Conditions with Recommended Design--High Flows
1:54.63 Scale Model