428

Rhone

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF YANHEE DAM SPILLWAY YANHEE PROJECT, THAILAND

Hydraulic Laboratory Report No. Hyd-428

DIVISION OF ENGINEERING LABORATORIES

COMMISSIONER'S OFFICE
DENVER, COLORADO

June 24, 1957

CONTENTS

CONTENTS--Continued

PREFACE

Hydraulic model studies of the spillway for Yanhee Dam, Yanhee Project, Thailand, were conducted in the Hydraulic Laboratory of the Bureau of Reclamation at Denver, Colorado, during 1956.

The final plans, evolved from this study, were developed through the cooperation of the staffs of the Dams Branch and the Hydraulic Laboratory Branch. Messrs. Taweechai Mackaman, Chareuk Nonthalhum, and Chamnan Pradisvanij, Civil Engineer trainees from the Thailand Royal Irrigation Department, conducted the majority of the tests, compiled the data, and assisted in analyzing the results under the supervision of the Hydraulic Laboratory staff.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Commissioner's Office--Denver	Laboratory Report No. Hyd-428
Division of Engineering Laboratories	Compiled by: W. E. Wagner
Hydraulic Laboratory Branch	Checked and
Hydraulic Structures and Equipment Section	reviewed by: A. J. Peterka
Denver, Colorado	J. W. Ball
June 24, 1957	Submitted by: H. M. Martin

Subject: Hydraulic model studies of Yanhee Dam spillway--Yanhee Project, Thailand

SUMMARY

The hydraulic model studies discussed in this report were made to evaluate the flow conditions in the approach channel, the characteristics of the flow over the spillway crest, the flow distribution in the transition and tunnels, and the geometry of the flow leaving the flip buckets. The results and recommendations contained herein are based on tests conducted on a 1:77.37 scale model of the spillway, Figure 4.

As a result of the model studies, the preliminary designs of the right pier and the transition were modified to improve the flow distribution at the spillway entrance and in the tunnels. Also, the lips of the flip buckets were elevated and the right flip bucket was moved farther downstream to better distribute the flow in the river channel.

Flow in the approach channel was satisfactory except in the vicinity of the left pier where a depression in the water surface formed and created a flow disturbance near the pier nose, Figure 6A and C. Fourteen different designs of the left pier were tested, Figures 8, 9, and 10. Several of the pier designs eliminated the flow disturbance and raised the depressed water surface at the pier, Table 1, but the design shown in Figure 10 gave the best hydraulic performance for the least cost. The maximum depression of the water surface at the pier for maximum discharge was reduced to 3 meters and the flow disturbance at the pier was completely eliminated. Therefore, the design shown in Figure 10 was recommended for construction in the field. Figures 11 and 21A show the approach conditions in the vicinity of the recommended left pier for discharges of 6,000 and 2,000 cms (212,000 and 71,000 cfs).

The flow distribution in the transition immediately downstream from the spillway crest was, in general, satisfactory, Figure 12. However, at the point where the transition joined the inclined tapering

tunnel, there was a tendency at all discharges for the flow to separate from the tunnel walls and rise along the sides of the tunnel. At the maximum discharge of 6,000 cms, fins of water crossed over the crown of the tapered tunnel. To eliminate these undesirable flow characteristics, the transition was modified by using a long radius curve to replace the sharp intersection at the junction of the transition and tapered tunnel, Figure 13B. The modified transition improved the flow conditions in the tunnel, Figures 14A, 15, and 21C. Very thin fins of water formed at the sides of the tunnel at near-maximum discharges but did not cross over the crown of the tunnel.

The flow distribution in the vertical bend and horizontal tunnel was excellent at all discharges, Figure 15B.

Minor changes in the location and shape of the flip buckets were made to improve the distribution of the jets in the river channel. The left bucket was moved 25 meters downstream from its preliminary location, Figure 18, to distribute the flow from the two tunnels longitudinally along the river channel. Also, the radius of both buckets was increased from 25 to 36 meters and the bucket lips raised 2 meters from elevation 142 to 144 (approximately) to clear the river water surface, to provide more lateral spreading of the jets, and to steepen the jet trajectories, Figures 18, 19, 20, and 21D. These changes reduced the river surface drawdown at the powerhouse from 7.5 to 6.7 meters.

Extensive model data for the recommended spillway design were obtained to aid in operating the structure. These data included discharge capacity curves, Figure 27; water surface profiles, Figures 22 and 23; the river surface drawdown curve, Figure 17; and piezometric pressures on the spillway crest and in the transition, Figures 24 and 25.

The model showed that certain gate combinations were best for releasing flows through the spillway. All four radial gates should be opened equal amounts to obtain the best flow conditions in the tunnels, in the buckets, and in the river channel. Satisfactory flow conditions occurred in the tunnels and in the buckets when releases were made through one tunnel, or with unequal flow in both tunnels, provided the two gates controlling the flow to each tunnel were opened equal amounts. Single gate operation produced poor flow conditions and should be avoided except in emergencies.

INTRODUCTION

Yanhee Dam site is located on the main stem of the Ping River about 50 kilometers (31 miles) upstream from Tak in the northwestern part of Thailand, and approximately 420 kilometers (260 miles) north of

Bangkok by air, Figure 1. A concrete arch type dam with a crest length of approximately 470 meters (1,542 feet) and a height of 154 meters (505 feet) above bedrock will impound a reservoir having a maximum water surface area of 3 million square meters (116 square miles) and a capacity of 12,200 million cubic meters (9,890,000 acre-feet).

The primary purpose of the multipurpose Yanhee Project is to produce power for use in the main population centers of Bangkok and Thonburi, and in 33 other changwads (Government provinces) of central and northern Thailand. Also, the dam and reservoir will provide a means for flood control, increased irrigation, and improved navigation along the Ping and Chao Phaya Rivers.

Power from the associated powerplant, with an ultimate installed capacity of 560,000 kilowatts, will be made available to 35 changwads by means of a transmission grid throughout central and northern Thailand. The project will also provide water for dry season irrigation of 2,300,000 rais (920,000 acres) of the Chao Phaya Project and make possible the development of 800,000 rais (320,000 acres) of new lands.

The spillway was designed for a maximum discharge of 6,000 cubic meters per second (212,000 second-feet) which will be controlled by 4 radial gates, each 11 meters wide by 17.4 meters high (36 by 57 feet). Water will spill over the concrete overflow crest, 44 meters (144 feet) in length, into two concrete-lined tunnels 375 meters (1,230 feet) and 396 meters (1,300 feet) in length and 11.3 meters (37 feet) in diameter, Figures 2 and 3. The spillway tunnels will discharge into the river channel, where the erosive effect of the floodwater will be minimized by flip buckets at the downstream ends of the tunnels.

Hydraulic model studies were conducted to investigate the approach conditions to the spillway, the flow characteristics of the spillway overflow section and tunnels, and the effect of the spillway flow on the downstream river channel.

THE MODEL

The model, constructed to a geometrical scale of 1:77.37, included a head box and tail box connected by the two spillway tunnels, Figure 4. Both head box and tail box were constructed of wood and lined with sheet metal. The spillway tunnels were constructed of transparent plastic to permit observation of the flow through the tunnels.

The head box contained the portion of the reservoir and topography for a distance of 165 meters upstream from the spillway crest. The topography was reproduced in the head box by placing a thin layer of mortar over metal lath tacked to wooden contours. The overflow crest section upstream from the tunnel portals was reproduced in concrete finished to a smooth surface using metal templates as guides. The spillway piers were made from wood, and the radial gates were shaped in sheet metal. Piezometers were placed in the overflow section and in the left transition and tunnel to study the pressure distribution in the structure.

The tail box contained the downstream tunnel portals, the flip buckets, and the river channel and adjacent topography for a distance of 550 meters downstream from the flip buckets. As in the head box, the topography was reproduced by placing a thin layer of mortar over metal lath and wooden contours, Figure 5A. To study the erosive effects of the spillway flow, the downstream river channel was molded in loose sand having a mean diameter of approximately 1 mm.

Water was supplied to the model by a centrifugal pump and measured by venturi meters which had been accurately calibrated in the laboratory. The completed model is shown in Figure 5B.

THE INVESTIGATION

General

The spillway studies concerned the entrance conditions to the spillway, the distribution of flow in the tunnels, the performance of the flip buckets, and the action of the flow as it entered the downstream river channel. In general, the investigation was conducted by first studying and modifying the upstream reaches of the spillway and then making detailed studies of the downstream portions of the structure. Thus, adverse flow conditions in the spillway entrance were corrected before extensive studies of the spillway tunnels or flip buckets were undertaken.

Although the model was operated through the entire range of discharges, most of the development studies were conducted using the normal maximum discharge of 2,000 cms and the maximum design discharge of 6,000 cms (70,700 and 212,000 second-feet, respectively). These discharges were considered sufficient to show that performance was satisfactory at all flows.

Operation of the Preliminary Design

In general, the operation of the preliminary design was satisfactory. However, it seemed desirable to make refinements in the spillway approach near the left pier and in the transitions downstream from the spillway crest. The reservoir topography and the spillway approach channel are such that the flow approaches the spillway in almost a direct line, Figure 2. However, for discharges near the maximum, some of the flow entered the spillway channel from the left along the upstream face of the arch dam, and caused a flow disturbance and depressed water surface at the left pier, Figure 6A and C.

The flow conditions at the right pier were very good at all discharges; the water surface between the piers in the right tunnel was comparatively level and a slight flow disturbance occurred at the right pier only at the maximum discharge, Figure 6A.

In general, the flow distribution in the tunnel entrance transitions was satisfactory, Figure 12. Flow conditions were similar for both tunnels and the water surfaces were comparatively level at corresponding cross sections within the transitions. The curvature of the water surface through each transition was approximately the same as the curvature of the tunnel invert indicating a uniform depth of flow. At the junction of the transition and the inclined tapered tunnel, however, the flow tended to separate from the tunnel side walls and there was an appreciable drop in the water surface which, when viewed from the side of the tunnel, gave a "humped" appearance of the water surface entering the tapered section, Figure 12A. This humped water surface occurred only at the sides of the tunnel; the water surface in the center of the tunnel curved uniformly downward. Although the two tunnels were identical, the humped water surface was more pronounced in the left tunnel where fins of water formed and crossed over the crown of the tapered tunnel at maximum discharge. The fins of water also were evident to a lesser degree in both tunnels for the lower range of discharges, Figure 12B.

The distribution of flow in the vertical bend and in the horizontal section of each tunnel was excellent, Figure 15. The radius of the vertical bend was ample as evidenced by the comparatively level water surface at any section in the bend and in the downstream tunnel.

Excellent flow distribution was also evident in the flip buckets near the downstream portals of the tunnels. On leaving the buckets, the jets spread adequately, even to the extent of striking the 1/4:1 slope of the rock cut at the right of the flip buckets, Figure 16. The trajectories of the jets were comparatively flat, and a slow eddy which caused no objectionable flow problems formed in the river channel on the left side of the jets.

Spillway Pier Studies

Right pier. The flow distribution in the vicinity of the right pier was very good and only a small unobjectionable flow disturbance was observed on the right side of the approach channel, Figure 6A. Therefore, no modifications to the right pier were made.

Left pier. Because of the proximity of the arch dam at the left of the spillway entrance, Figure 2, part of the flow turned abruptly through an angle of about 90° to enter the spillway crest section. The sharp turn caused an undesirable flow disturbance at the left pier, Figure 6C. The unsightly flow disturbance, in addition to reducing the discharge coefficient of the spillway, caused an uneven flow distribution in the left spillway transition and tunnel. Exploratory testing indicated that a large curved pier extending into the reservoir was necessary to reduce the flow disturbance, Figure 7.

Extensive testing, using 14 different designs, was undertaken to eliminate the flow disturbance at the left pier and improve the flow distribution in the tunnels. The first five designs, Designs B through F, Figure 8, were curved walls of various shapes and lengths extending into the reservoir. In general, the flow disturbance near the crest was eliminated, but a smaller flow disturbance occurred at the upstream end of the walls except Design B which was elliptical in general shape. This wall produced good flow conditions and completely eliminated the flow disturbance, Figure 7B. Testing was continued, however, to reduce the wall length and refine the design.

Wall Designs G through N, forming smaller elliptical or parabolic piers, Figure 9, approximated the shape of Design B. Each of these designs eliminated the flow disturbance and were evaluated by measuring the amount of the water surface depression along the inside face of the pier. The depression was measured as the maximum vertical distance in meters between the reservoir surface and the depressed water surface along the face of the pier for the maximum discharge of 6,000 cms.

The maximum depression for Designs G, H, J, and K was 6.1, 5.3, 5.0, and 4.5 meters, respectively. The depression measured with the elliptical shapes of Designs G and H were slightly larger than the parabolic shapes of Designs J and K. To further reduce the size of the left pier and alleviate the depressed water surface, pier shapes having elliptical or parabolic inside faces and small circular arcs on the outside faces were tested, Designs L, M, and N, Figure 9.

Designs L and M, having elliptical inside faces, reduced the depression to 3.6 and 3.8 meters, respectively. Design L, which had a 3-meter circular arc on the outside face, gave a fairly good flow pattern with no flow disturbance along the pier face. The 1-meter circular arc on Design M was insufficient to provide smooth flow around the pier nose and a slight flow disturbance was observed at the pier nose.

From the above tests, it appeared that an intermediate circular arc would provide sufficient curvature at the pier nose and that a parabolic inside face would be less blunt and provide a more gradual change in direction than the elliptical shaped piers. Therefore, Design N included a 2-meter circular arc and a parabolic curve joining the circular arc with the training wall, Figure 9. The flow pattern along the face of Pier Design N was very good; however, there was a small flow disturbance at the break in curvature where the parabolic curve joined the training wall.

In the recommended pier design, the break in curvature was eliminated by placing a 10-meter radius circular arc tangent to the parabolic curve and the training wall, Figure 10. The flow was smooth and appeared to accelerate uniformly with no flow disturbances along the face of the pier, Figure 11C. The maximum depression was 3.0 meters and occurred well upstream from the spillway crest. A summary of the left pier studies is given in Table 1.

Although the recommended pier improved the flow distribution in the tunnel transitions, the flow still tended to climb the sides of the tunnels. Thus, the undesirable flow distribution in the transitions was not entirely due to the poor approach conditions observed at the left pier during the preliminary studies. It was decided that modification of the transitions would be required to improve the flow distribution in the tunnels.

Transition Studies

The humped water surface and the tendency for the flow to separate at the junction of the transition and the circular tunnel noted for near-maximum flows during the pier studies, Figure 12A, were also observed at intermediate discharges both for free flow and partial gate openings, Figure 12B. The tendency for flow separation and the "humping" of the water surface were reduced slightly by installing the recommended left pier. That the adverse flow conditions in the transition were a result of the transition shape and not primarily due to the poor approach conditions in the vicinity of the left pier was demonstrated by placing a long wall, projecting about one hundred meters into the reservoir, along the left side of the spillway approach. Although the lengthy wall provided ideal approach conditions, it did not eliminate the adverse flow distribution in the transitions.

A study of the preliminary design transition, Figure 13, indicated a possible solution to the problem. The boundary formed by the sides of the transition had an abrupt change in alinement where the transition joined the circular tunnel, Figure 13A. It appeared that replacing the abrupt change in alinement in the boundary with a smooth curve would cause the flow to follow the boundary and reduce or eliminate the humped water surface. The transitions were modified by using a circular arc, with a radius of 164 meters, to form a smooth curve at the sides where the transitions joined the tapered tunnels, Figure 13B.

The flow conditions in the modified transitions were improved. No tendency for the flow to separate from the tunnel side walls was observed, and the humped water surface was only slightly evident at maximum discharge, Figure 14A. Very thin fins of water formed at the sides and rose toward the crowns of the tunnels. The fins of water had little thickness and did not cross over the crowns of the tunnels so were not considered objectional.

Other minor modifications to the spillway approach and piers were studied in an attempt to further improve the flow conditions in the transition, but none of these modifications reduced the height of the fins. One modification reduced the length of the piers which extended downstream into the tunnel transitions. Figure 14B shows the flow conditions in the transition with the center pier in the left tunnel shortened about 2 meters at the tunnel invert. This change caused the fins of water to rise higher on the sides of the tunnel and created an additional fin in the center of the tunnel downstream from the pier. Therefore, the shortened pier was not considered acceptable and the center piers shown in the preliminary design were proposed for the prototype.

It became apparent that major changes in the transition shape, such as increasing the transition length, would be necessary to completely eliminate the fins at the sides of the tunnels. Because the fins of water did not cross over the crown of the tunnel at any discharge, and because they became smaller in height as the discharge decreased, the modified transition design shown in Figure 13B was considered adequate and was recommended for construction.

Flow in Tunnels

The flow distribution in the tunnels downstream from the transitions was excellent for all discharges, as evidenced by the comparatively level water surface at any cross section in the vertical bends and in the horizontal tunnels, Figure 15. Therefore, no changes were deemed necessary in these portions of the spillway.

Flip Bucket Studies

The distribution of flow in the preliminary flip buckets was excellent as shown by the uniform thickness of the jets leaving the buckets, Figure 16B and C. The jets on leaving the flip buckets appeared to spread adequately, and there was no objectionable interference between the two jets which joined near the high point of the trajectories. However, the edge of the right jet struck the 1/4:1 slope of the excavated channel and could not continue to spread laterally. Also, an unstable flow condition occurred at intermediate discharges. The water surface in the river was higher than the bucket lips and the excavated channels downstream were alternately filled with water and swept out in a rhythmic motion. The trajectories of the jets were comparatively flat and caused a considerable lowering of the water surface in the tailrace of the powerhouse. At the maximum discharge of 6,000 cms, the drawdown was 7.5 meters (24.5 feet), Figure 17. In this study drawdown is defined as the difference in elevation between the water surface at the powerhouse tailrace and in the river channel about 500 meters downstream from the flip buckets.

To prevent the right jet from striking the 1/4:1 slope of the excavated channel, the right bucket was moved 25 meters downstream from its preliminary location, Design A, Figure 18. This change permitted the right jet to spread laterally and strike a wider portion of the river channel. It also separated the areas of impact of the two jets and spread the spillway flow over a longer reach of the river. The drawdown of 7.2 meters at maximum discharge was only slightly less than the 7.5 meters observed with the preliminary design, Figure 17.

It was desirable to reduce this drawdown in the powerhouse tailrace and this could be done by steepening the trajectories of the jets. The radius for the buckets was therefore increased from 25 to 36 meters and the bucket lips were raised from elevation 142 to 144, Figures 18 and 19. This change raised the bucket lips above the water surface in the river channel and eliminated the unstable flow conditions observed with the preliminary buckets. The jet trajectories were slightly steeper and the drawdown was reduced to 6.7 meters, recommended design, Figure 17. Also, the lateral spread of the jets was greater than the spread observed in the preliminary design, Figure 20. There were no objectionable return eddies along the sides of the river channel.

THE RECOMMENDED SPILLWAY

The recommended spillway, including the final left pier design, Figure 10, the modified transition, Figure 13B, and the modified flip bucket, Figure 19, is shown in Figure 3. The operation of the recommended spillway at discharges of 2,000 and 6,000 cms is shown in Figures 11, 15, 20, and 21.

Considerable model data were obtained from the recommended spillway to predict the performance of the prototype structure and to determine the best operating procedures.

Water Surface Profiles

Profiles of the water surface at each of the five piers and in the left spillway tunnel were obtained for total discharges of 2,000 and 6,000 cms through both tunnels, Figure 22. The water surface profiles show the amount the water surface was depressed at the piers and the uniform distribution of flow in the tunnels. Figure 23 shows the geometry of the spillway flow downstream from the flip buckets.

Pressure Measurements

Piezometers were installed in critical regions of the spillway crest and in the left tunnel transition to make certain that the shapes of the crest and transition were hydraulically satisfactory. Pressures were observed for free flow at discharges of 2,000 and 6,000 cms with equal flow in both tunnels, Figure 24, and for partial gate openings of 2, 4, 8, and 12 meters with the reservoir surface at maximum elevation, Figure 25.

For free flow, the pressures on the spillway crest and on the invert of the transition were above atmospheric for discharges of 2,000 and 6,000 cms. The lowest pressure observed on the spillway crest was 1.8 meters below atmospheric at Piezometer 8 for a 4-meter gate opening, Figure 25. Subatmospheric pressures of about 0.7 meter were also observed at Piezometer 8 for 2- and 8-meter gate openings. The pressures along the base of the left training wall (Piezometers 17-20) were above atmospheric except for the 2-meter gate opening when pressures ranging from 0.3 to 0.5 meter below atmospheric were observed.

The lowest observed pressure on the transition invert occurred at Piezometer 40 near the downstream end of the transition and was 0.6 meter above atmospheric at the maximum discharge of 6,000 cms, Figure 24. However, subatmospheric pressures were observed at Piezometers 27 and 28 on the left side of the transition, and at Piezometer 34 in the corner formed by the invert and left side of the transition, Figure 24. The lowest observed pressure was 2.8 meters below atmospheric at Piezometer 28. Although the pressure at Piezometer 28 is comparatively low, it is well above the cavitation range. None of the subatmospheric pressures were considered critical.

Erosion Studies

To help evaluate the hydraulic performance of the preliminary and recommended spillway design, erosion tests were made with each design at the maximum discharge of 6,000 cms, Figure 26. In each case, the model was operated for a time period of 1-1/2 hours, which was equal to about 13 hours of prototype operation. A direct comparison between the two designs cannot be made because the sand level in the river channel was at elevation 136 meters before the preliminary design test, and at elevation 128 before the recommended design test. However, the eroded area upstream from the flip buckets is considerably smaller for the recommended design. A large, shallow scour pocket formed downstream from the flip buckets near the right riverbank when the recommended design was installed indicating a better lateral spreading of the jets from the buckets.

It should be noted that the model erosion tests are only qualitative. They predict only where erosion might occur and not the erosion depth which might be expected in the prototype.

Spillway Calibration

Spillway rating curves showing the relation of reservoir elevation and discharge for free flow and partial gate openings were determined from model data, Figure 27. The curves for free flow and for gate openings of 2-meter increments were determined by calibration tests, while the intermediate gate openings were determined by interpolation and spot-check calibration tests. The gate opening was measured as the difference in elevation between the spillway crest and the bottom of the gate.

The spillway coefficient curve for free flow was plotted against reservoir elevation, Figure 27. "C" was computed from the equation, $Q = CLH^{3/2}$, for both metric and English systems. In this equation Q is the discharge in cms or cfs, C is the coefficient of discharge, L is the length of crest between piers in meters or feet, and H is the head above the crest in meters or feet. A coefficient C, of 1.98 (metric) and 3.44 (English) occurred at the maximum reservoir elevation 260.5.

Gate Operating Procedure

The model tests showed that the best flow distribution occurred in the river channel when the spillway flow was released through both tunnels with all gates opened equal amounts, Figures 20B and 21B. The flow spread over about two-thirds of the river channel, and side eddies were at a minimum. Likewise, good flow distribution occurred in the river channel when discharges less than 3,000 cms were released through either tunnel, if the pair of tunnel gates controlling the flow were opened equal amounts, Figure 28. The spillway flow with either one of the tunnels operating covered about half of the river channel; side eddies were not objectionable. However, if releases are made through one gate, poor flow distribution will occur in the tunnel and flip bucket, Figure 29. Part of the flow will spiral over the crown of the tunnel and concentrate on one side of the flip bucket.

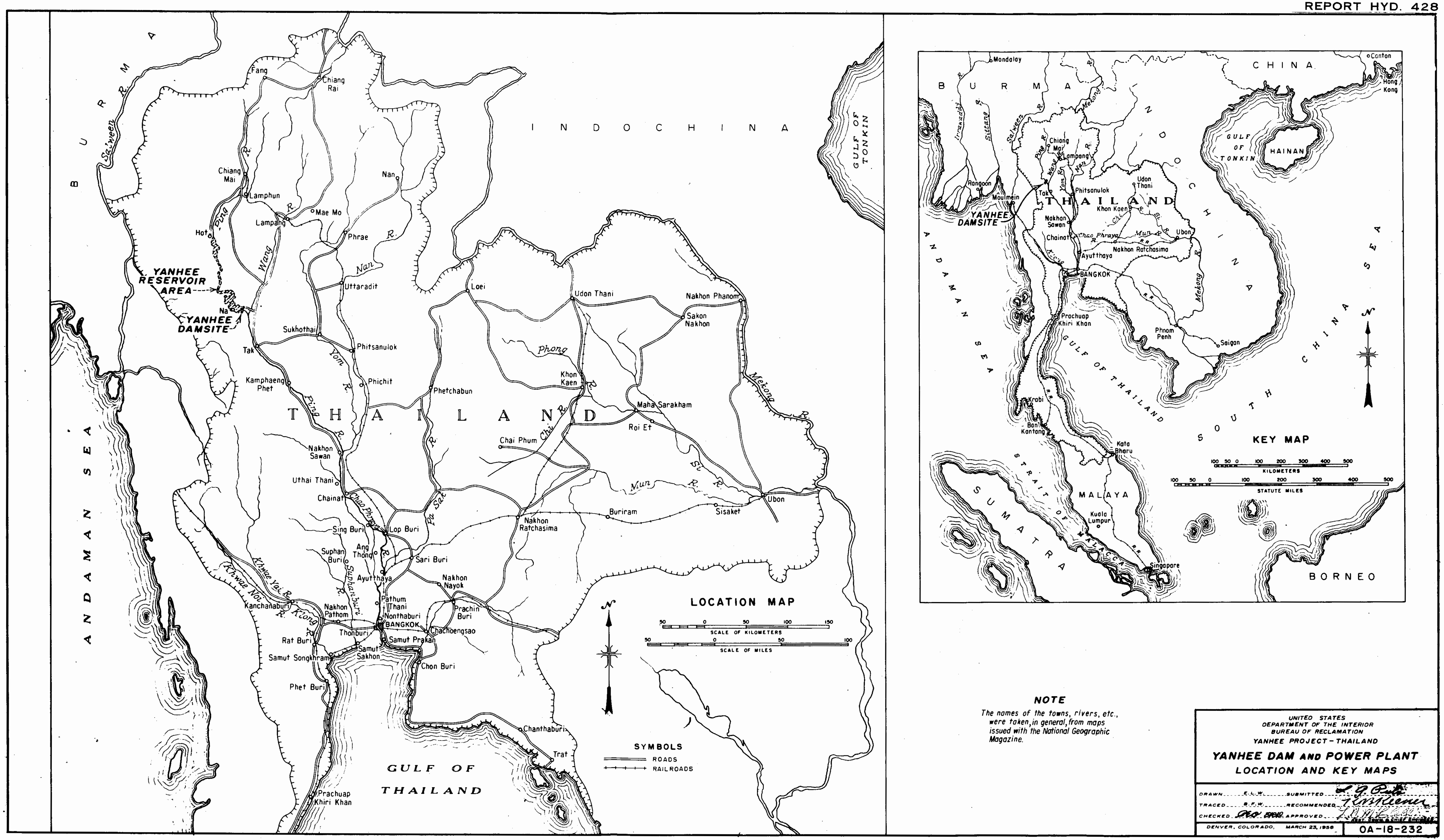
For best performance of the spillway, it is recommended that flows be released through both tunnels with all gates equally open. Flows also may be released through one tunnel if the two gates controlling the flow to that tunnel are opened equal amounts. Operation of either spillway tunnel with unequal gate openings should be avoided and used only in emergencies.

Table 1

LEFT PIER STUDIES

Pier	Depression*	Remarks
Prelim design	Not measured	Flow disturbance at pier
B	Not measured	Fairly good flow pattern
C D	Not measured	Flow disturbance at upstream end of curved wall
E	Not measured	Flow disturbance at sloping top of wall
F	Not measured	Insufficient curvature at pier nose
G H	6.1 5.3	Curvature too great in vicinity of pier nose
J K	5.0 4.5	Flow disturbance where parabola joins training wall
L	3.6	Good flow pattern
M	3.8	Slight disturbance at pier nose
N	Not measured	Flow disturbance where parabola joins training wall
Recom design	3.0	Very good flow pattern

*Maximum vertical distance in meters between the reservoir elevation and the depressed water surface along the face of the pier at the maximum discharge of 6,000 cms.



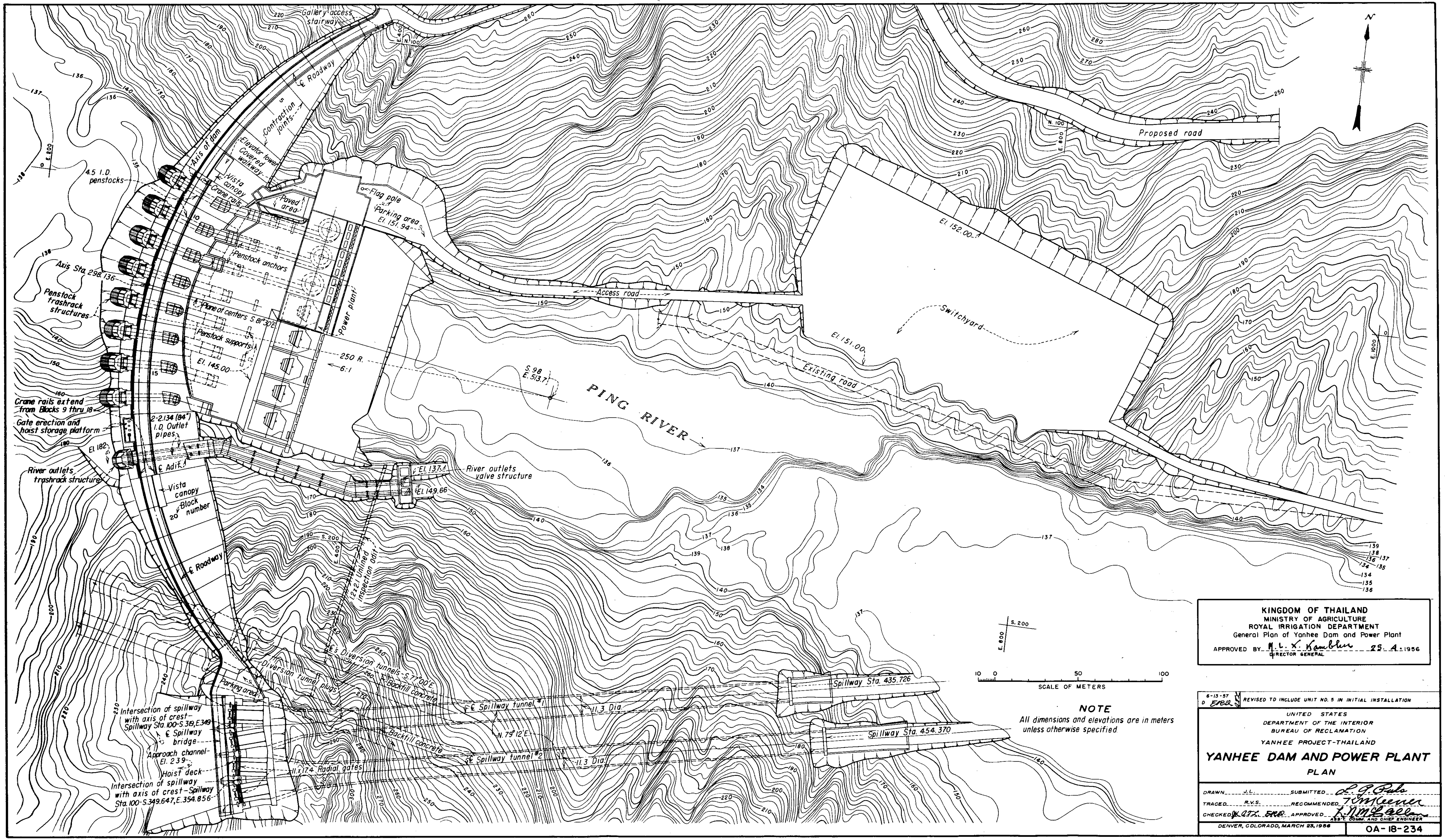
NOTE

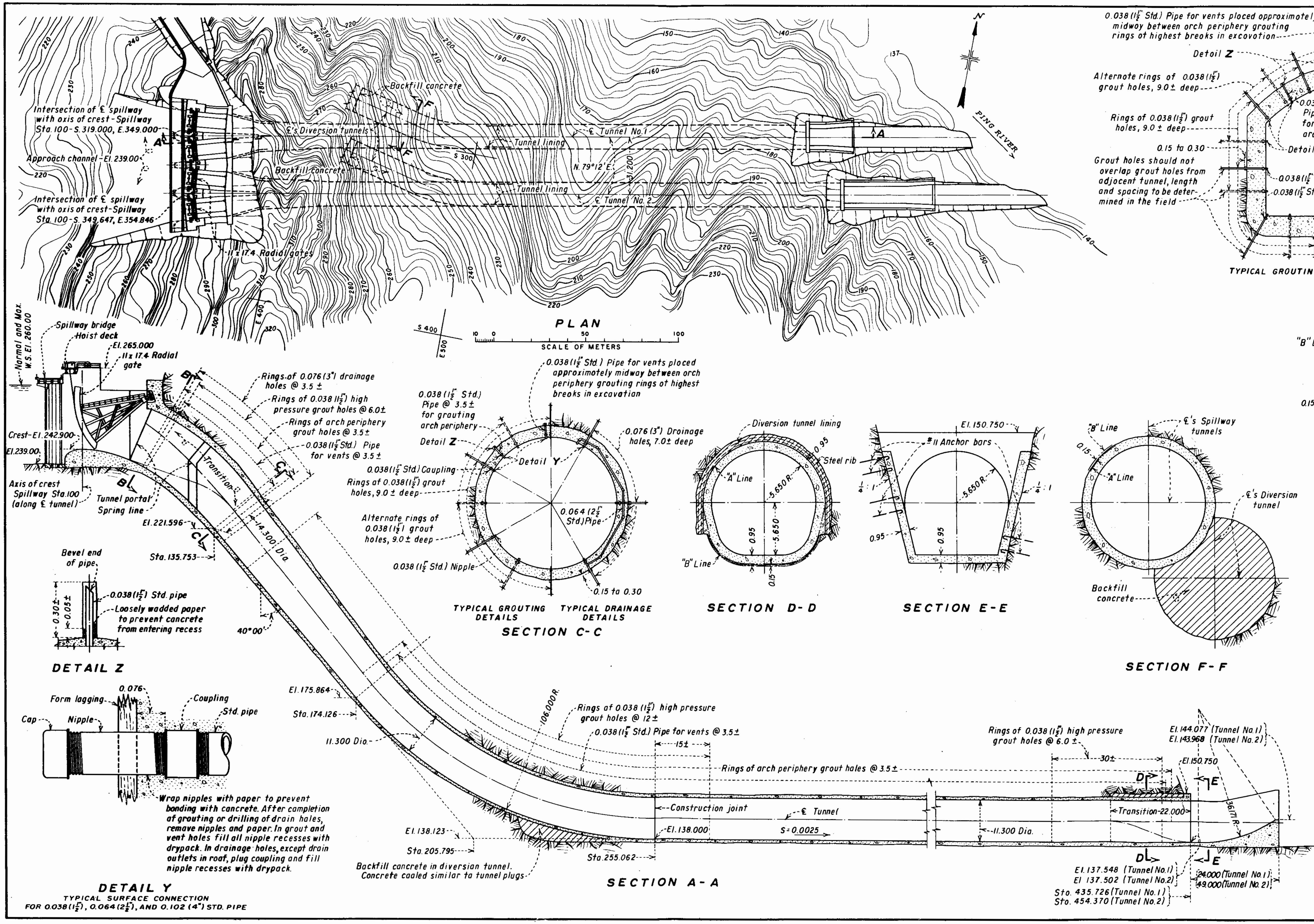
The names of the towns, rivers, etc., were taken, in general, from maps issued with the National Geographic Magazine.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YANHEE PROJECT – THAILAND

YANHEE DAM AND POWER PLANT
LOCATION AND KEY MAPS

DRAWN E.L.W. SUBMITTED
TRACED E.F.W. RECOMMENDED
CHECKED APPROVED
DENVER, COLORADO, MARCH 23, 1956
OA-18-232


KINGDOM OF THAILAND
MINISTRY OF AGRICULTURE
ROYAL IRRIGATION DEPARTMENT
General Plan of Yanhee Dam and Power Plant
APPROVED BY M. L. X. Kambhu 25-4-1956
DIRECTOR GENERAL
REVISED TO INCLUDE UNIT NO. 5 IN INITIAL INSTALLATION
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YANHEE PROJECT-THAILAND
YANHEE DAM AND POWER PLANT
PLAN
DRAWN J.L.
TRACED R.V.S.
SUBMITTED
RECOMMENDED
APPROVED
ASS'T. COMM. AND CHIEF ENGINEER
DENVER, COLORADO, MARCH 23, 1956
OA-18-234
NOTE
All dimensions and elevations are in meters unless otherwise specified
SCALE OF METERS
PING RIVER
Switchyard
Power plant
Access road
Existing road
Proposed road
Parking area
El. 151.94
El. 152.00
El. 151.00
Flag pole
Paved area
Gallery access stairway
℄ Roadway
Contraction joints
Elevator tower
Covered walkway
Vista canopy
Crane rails
Axis of dam
4.5 I.D. penstocks
Axis Sta. 298.136
Penstock trashrack structures
Penstock anchors
Penstock supports
Plane of centers S. 81°30'E.
El. 145.00
250 R.
Crane rails extend from Blocks 9 thru 18
Gate erection and hoist storage platform
2-2.134 (84") I.D. Outlet pipes
El. 182
River outlets trashrack structure
℄ Adit
River outlets valve structure
El. 137.4
El. 149.66
Block number
1.2x2. Unlined inspection adit
℄'s Diversion tunnels-S.77°00'E.
Diversion tunnel plugs
Backfill concrete
℄ Spillway tunnel #1
℄ Spillway tunnel #2
11.3 Dia.
N. 79°12'E.
Spillway Sta. 435.726
Spillway Sta. 454.370
Intersection of spillway with axis of crest-Spillway Sta. 100-S.319, E.349
℄ Spillway bridge
Approach channel-El. 239
Hoist deck
11 x 17.4 Radial gates
Intersection of spillway with axis of crest-Spillway Sta. 100-S.349.647, E.354.856



CONCRETE FINISHES

Tunnel below ℄ or spring line .... F4 or U3
Tunnel below ℄ from Sta. 172 to Sta. 258 .... F4 with special finish or U3
Interior surfaces of portal structure .... F4 or U3
All other surfaces .... F2 or U2

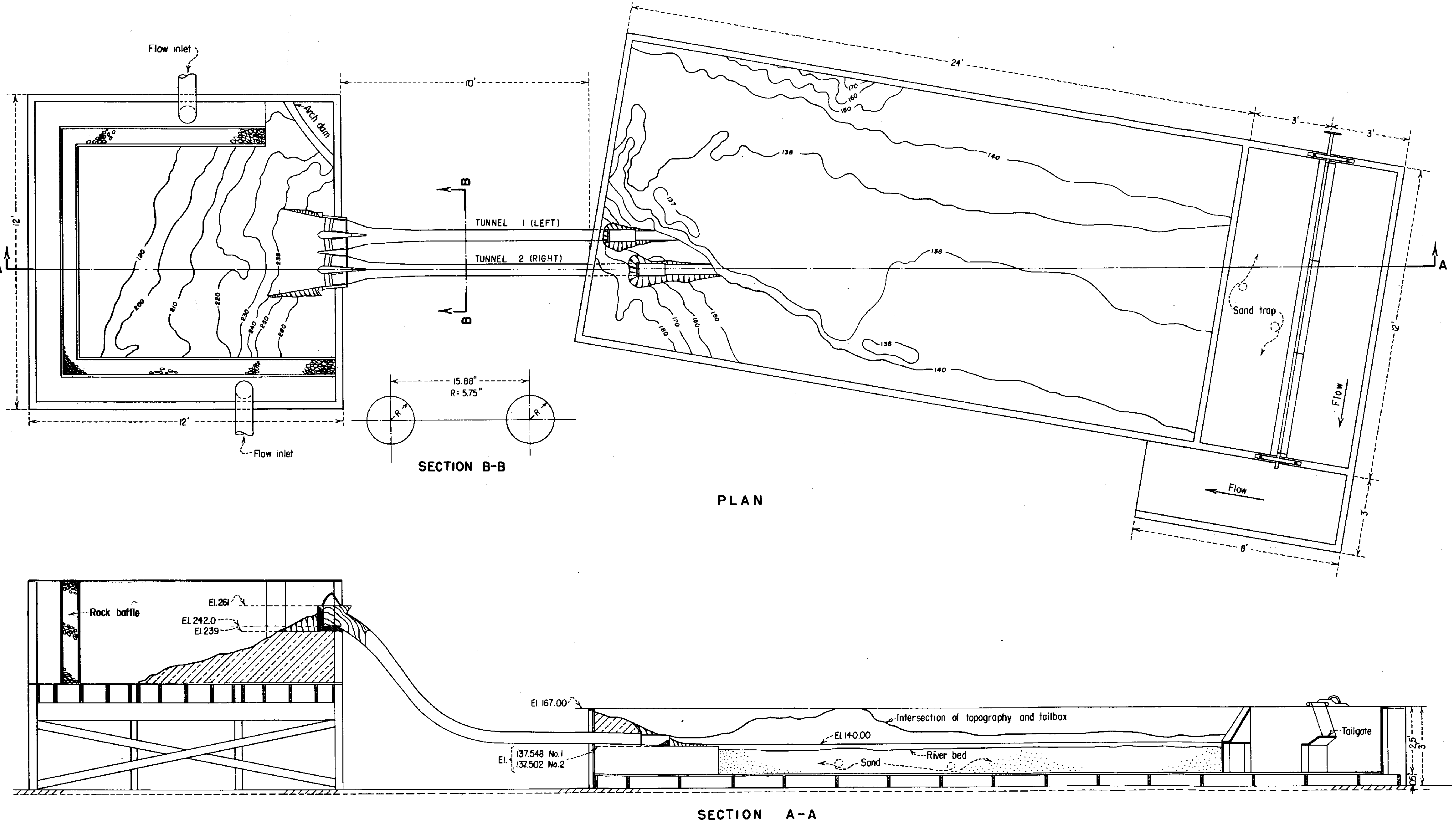
NOTES

Chamfer all exposed corners 0.020 unless otherwise specified
Grout holes to be drilled and grouted after periphery grouting is completed.
Drainage holes to be drilled after all grouting is completed.
All dimensions and elevations are in meters unless otherwise specified.
Reinforcement required but not shown.

5-24-57 ADDED TUNNEL SUPPORTS AT DOWNSTREAM END.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YANHEE PROJECT - THAILAND
YANHEE DAM
SPILLWAY
PLAN AND SECTIONS

DRAWN J.L. SUBMITTED L.G. Puls
TRACED E.W.C. RECOMMENDED
CHECKED JK&DJ APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MAY 1, 1956
0A-18-245



Note: All dimensions in feet or inches
Elevations in meters.

YANHEE DAM SPILLWAY
MODEL LAYOUT - PRELIMINARY DESIGN
1:77.37 SCALE MODEL

A. Placing topography in tail box.

B. The completed model.

YANHEE DAM SPILLWAY
The 1:77.37 Scale Model

A. Flow entering spillway.

B. Preliminary left pier.

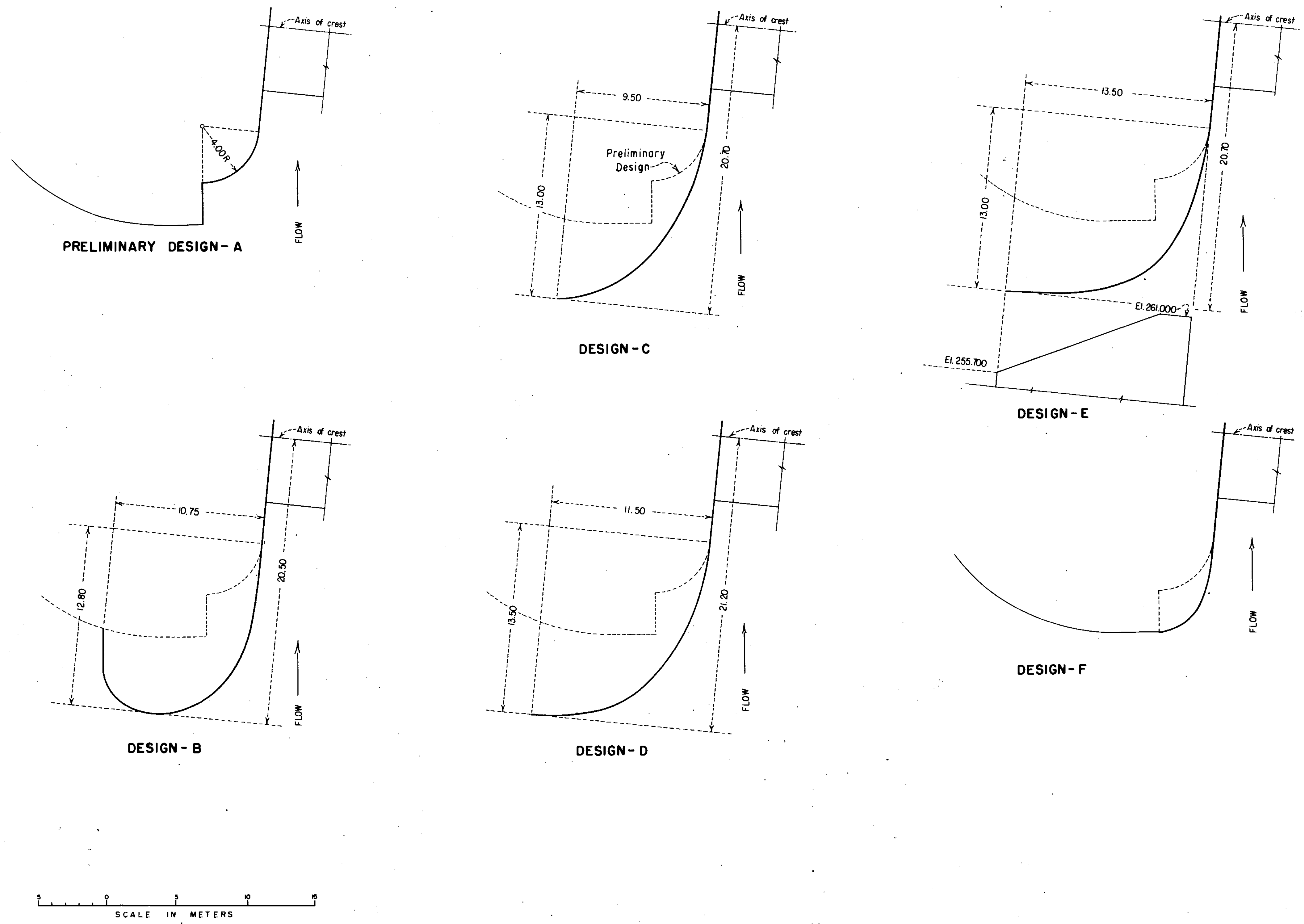
C. Flow disturbance at left pier.

YANHEE DAM SPILLWAY
Flow Conditions for Preliminary Pier Design
Maximum Discharge = 6000 CMS
1:77.37 Scale Model

A. Modified left pier nose and flow disturbance - Design A.

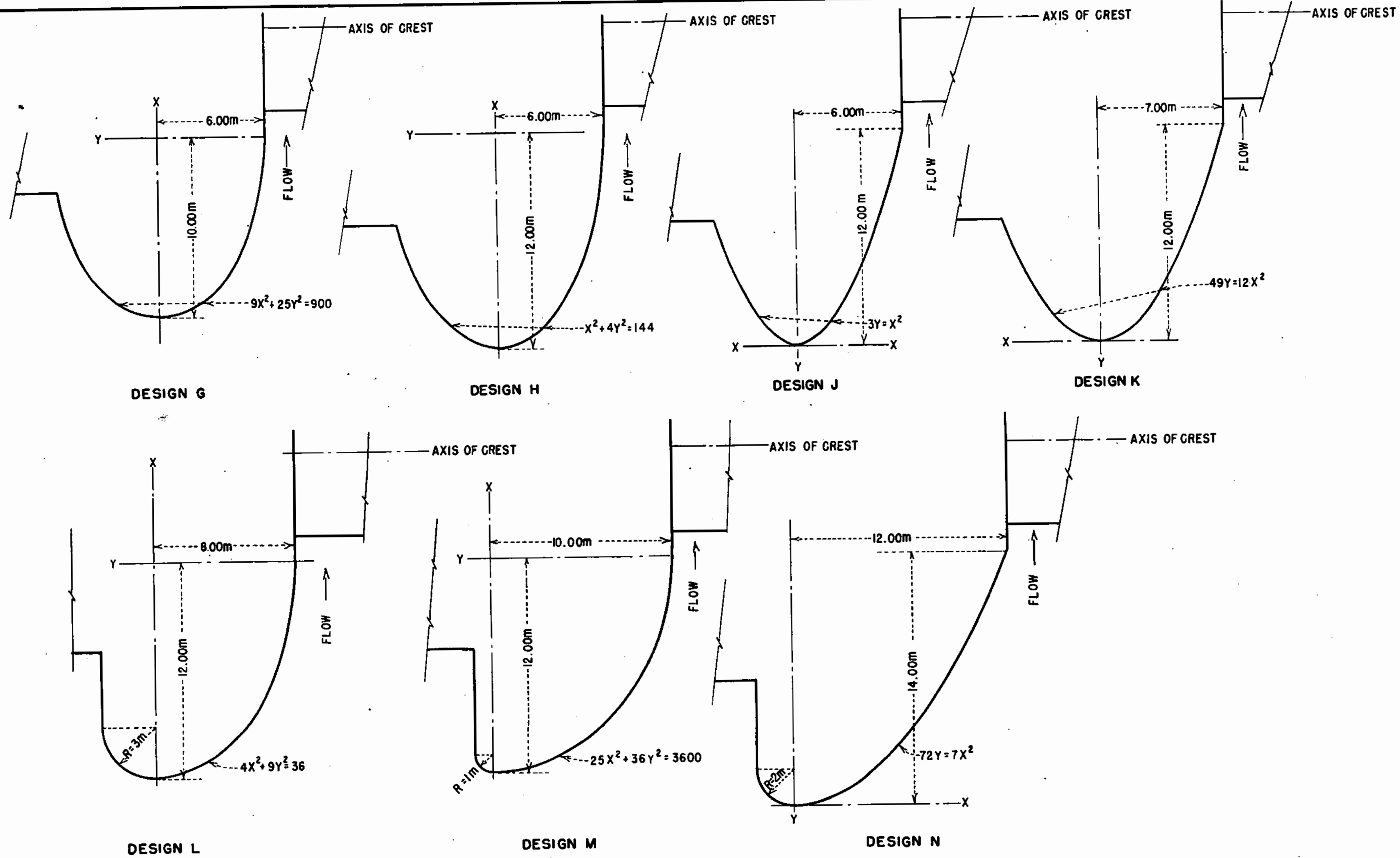
B. Design B.

YANHEE DAM SPILLWAY
Flow Conditions for Left Pier Designs A and B
Maximum Discharge = 6000 CMS
1:77.37 Scale Model



YANHEE DAM SPILLWAY
LEFT PIER DESIGNS A-F
1:77.37 SCALE MODEL

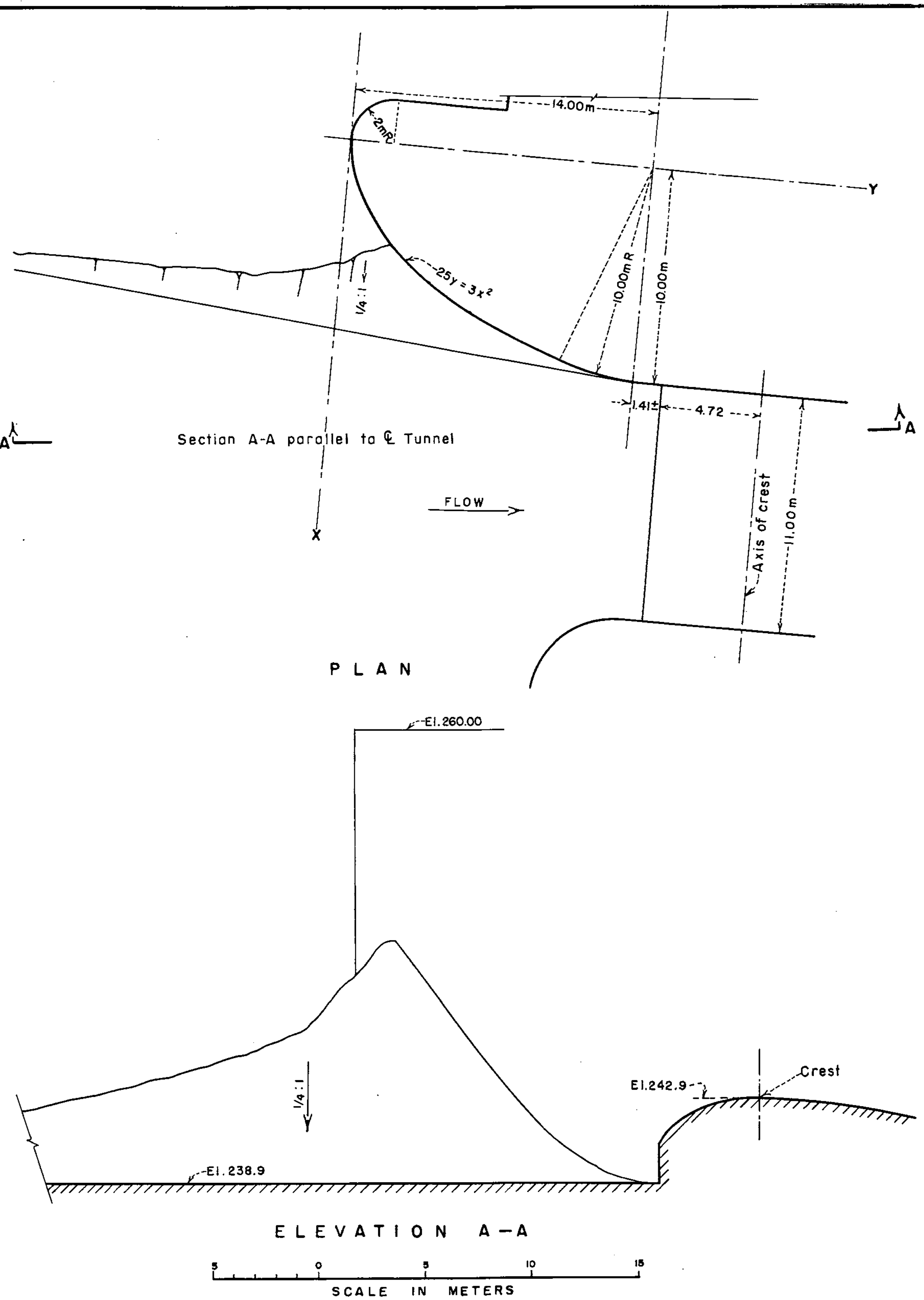
539



YANHEE DAM SPILLWAY
LEFT PIER DESIGNS G-N
1:77.37 SCALE MODEL

FIGURE 9
REPORT HYD. 428

FIGURE 10
REPORT HYD 428



YANHEE DAM SPILLWAY
RECOMMENDED LEFT PIER
1:77.37 SCALE MODEL

539

A. General approach conditions.

B. Spillway entrance.

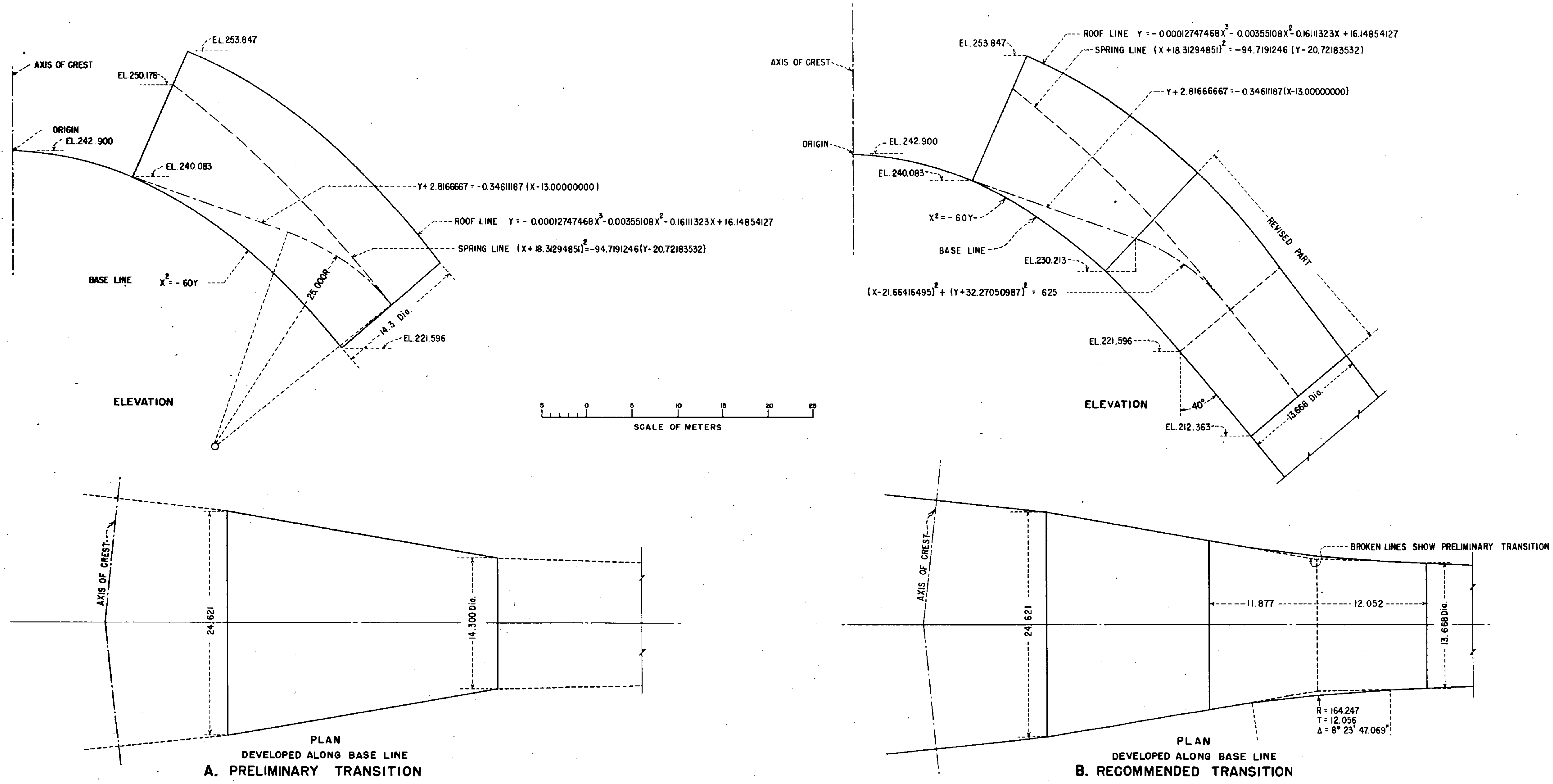
C. Flow at piers.

YANHEE DAM SPILLWAY
Flow Conditions for Recommended Piers
Maximum Discharge = 6000 CMS
1:77.37 Scale Model

A. Maximum discharge = 6000 CMS. Note hump in water surface at downstream end of transition.

B. Discharge = 2000 CMS. Gates partially closed at maximum reservoir elevation.

YANHEE DAM SPILLWAY
Flow Conditions in Preliminary Transition
1:77.37 Scale Model



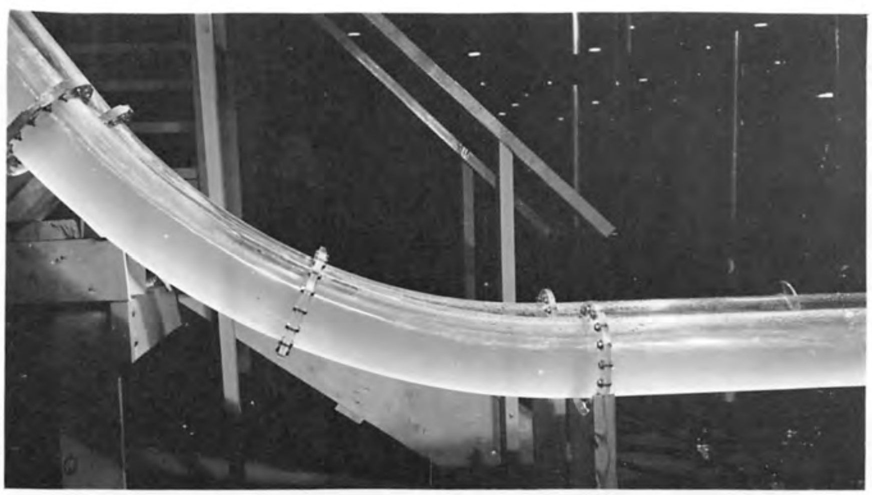
YANHEE DAM SPILLWAY
PRELIMINARY AND RECOMMENDED TRANSITIONS
1:77.37 SCALE MODEL

A. Recommended transition in left tunnel and preliminary transition in right tunnel. Note absence of fin of water in right tunnel.

B. Center pier in left tunnel shortened 2 meters. Note increase in height of fin.

YANHEE DAM SPILLWAY
Flow Conditions in Tunnel Entrance Transitions
Maximum Discharge = 6000 CMS
1:77.37 Scale Model

A. Flow through tunnels. Recommended transition in left tunnel and preliminary transition in right tunnel.



B. Flow in vertical bend.

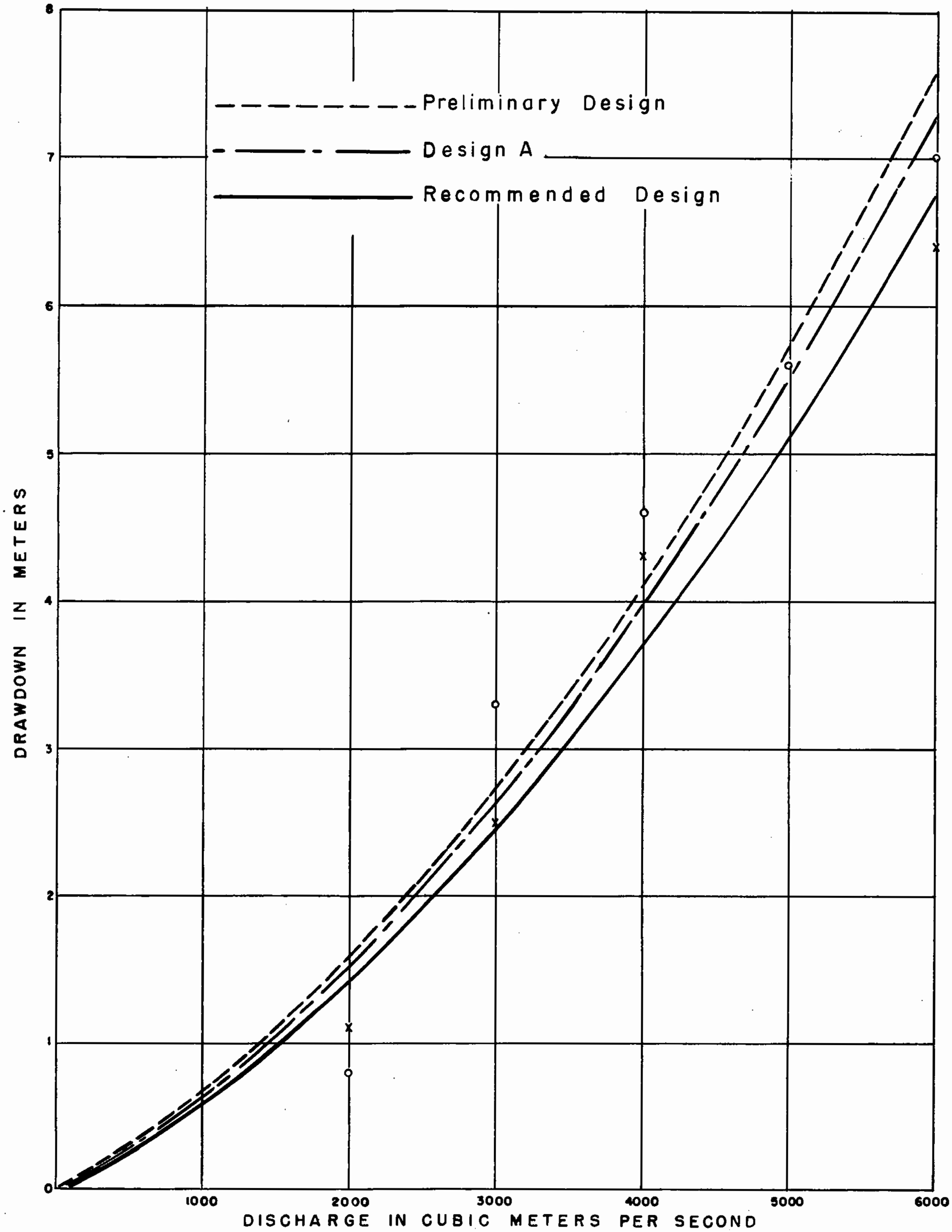
YANHEE DAM SPILLWAY
Flow Conditions Through Tunnels and Vertical Bend
Maximum Discharge = 6000 CMS Recommended Design
1:77.37 Scale Model

A. Preliminary flip buckets.

B. Flow leaving buckets.

C. Note right jet striking 1/4:1 slope.

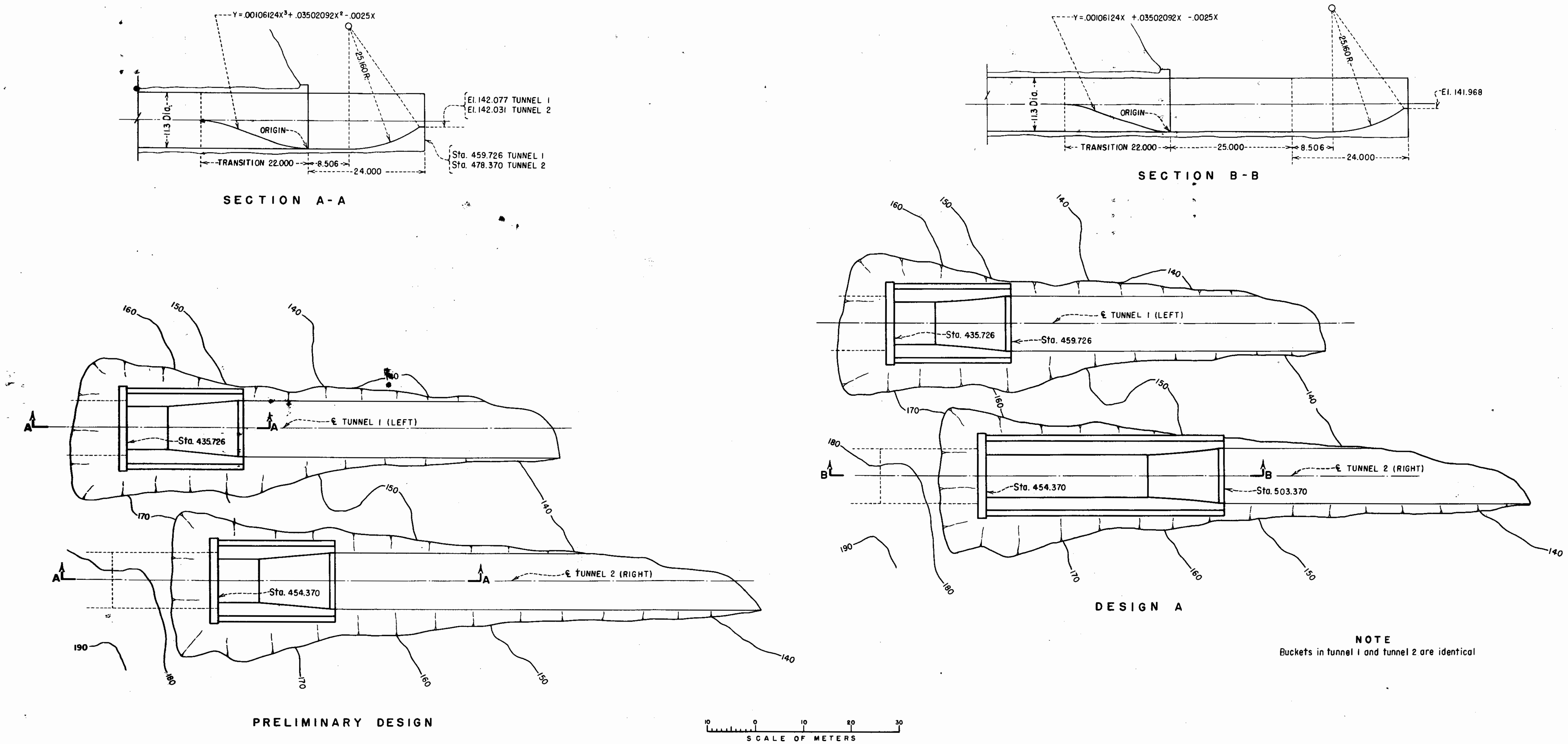
YANHEE DAM SPILLWAY
Flow Conditions for Preliminary Flip Buckets
Maximum Discharge = 6000 CMS
1:77.37 Scale Model



YANHEE DAM SPILLWAY

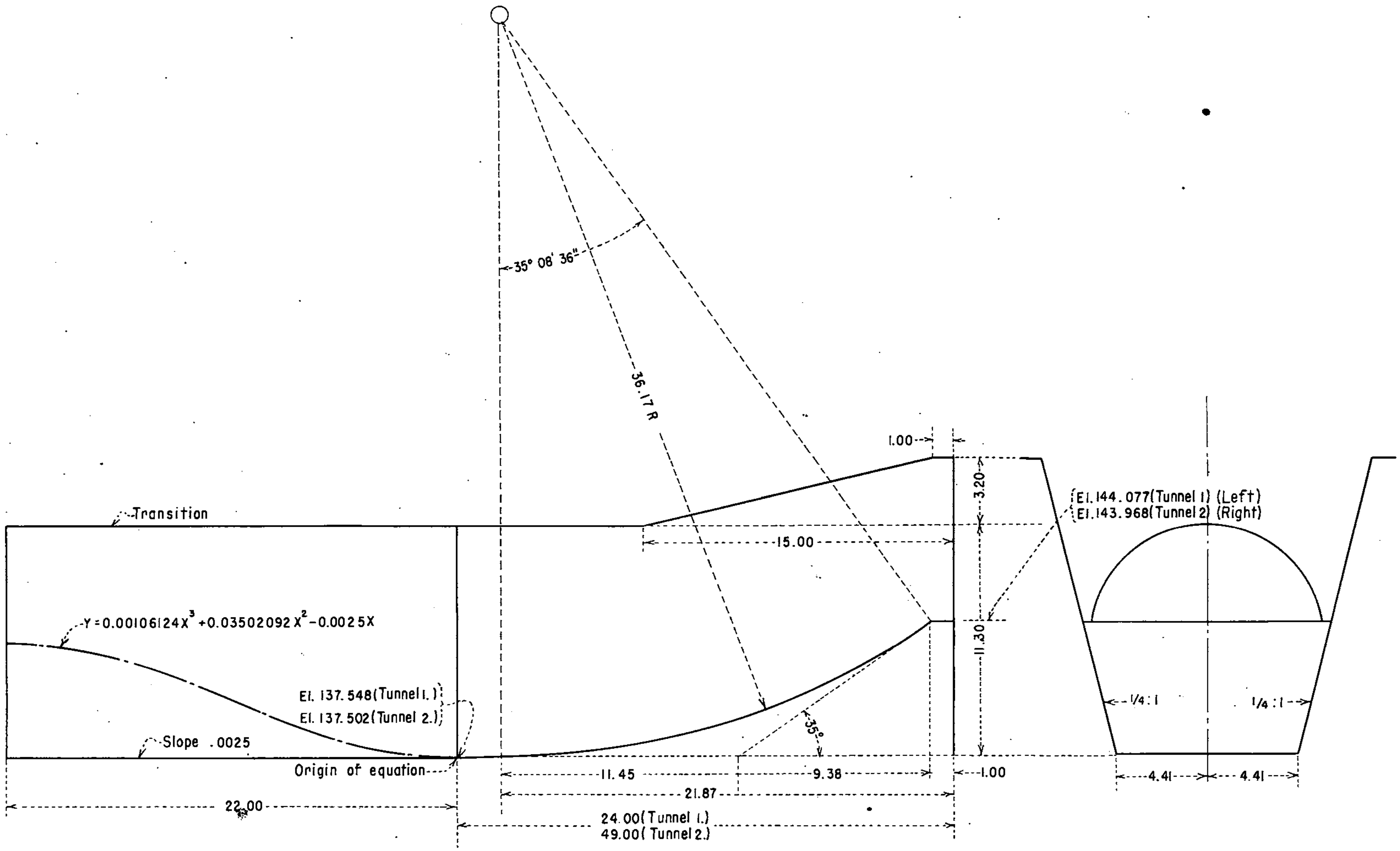
RIVER SURFACE DRAWDOWN CURVES FOR VARIOUS BUCKETS

1:77.37 SCALE MODEL



YANHEE DAM SPILLWAY
FLIP BUCKETS
PRELIMINARY AND DESIGN A
1:77.37 SCALE MODEL

539



FLIP BUCKET

YANHEE DAM SPILLWAY
RECOMMENDED FLIP BUCKETS
1:77.37 SCALE MODEL

FIGURE 19
REPORT HYD 428

A. Recommended flip buckets.

B. Flow leaving buckets

YANHEE DAM SPILLWAY
Flow Conditions for Recommended Buckets
Maximum Discharge = 6000 CMS
1:77.37 Scale Model

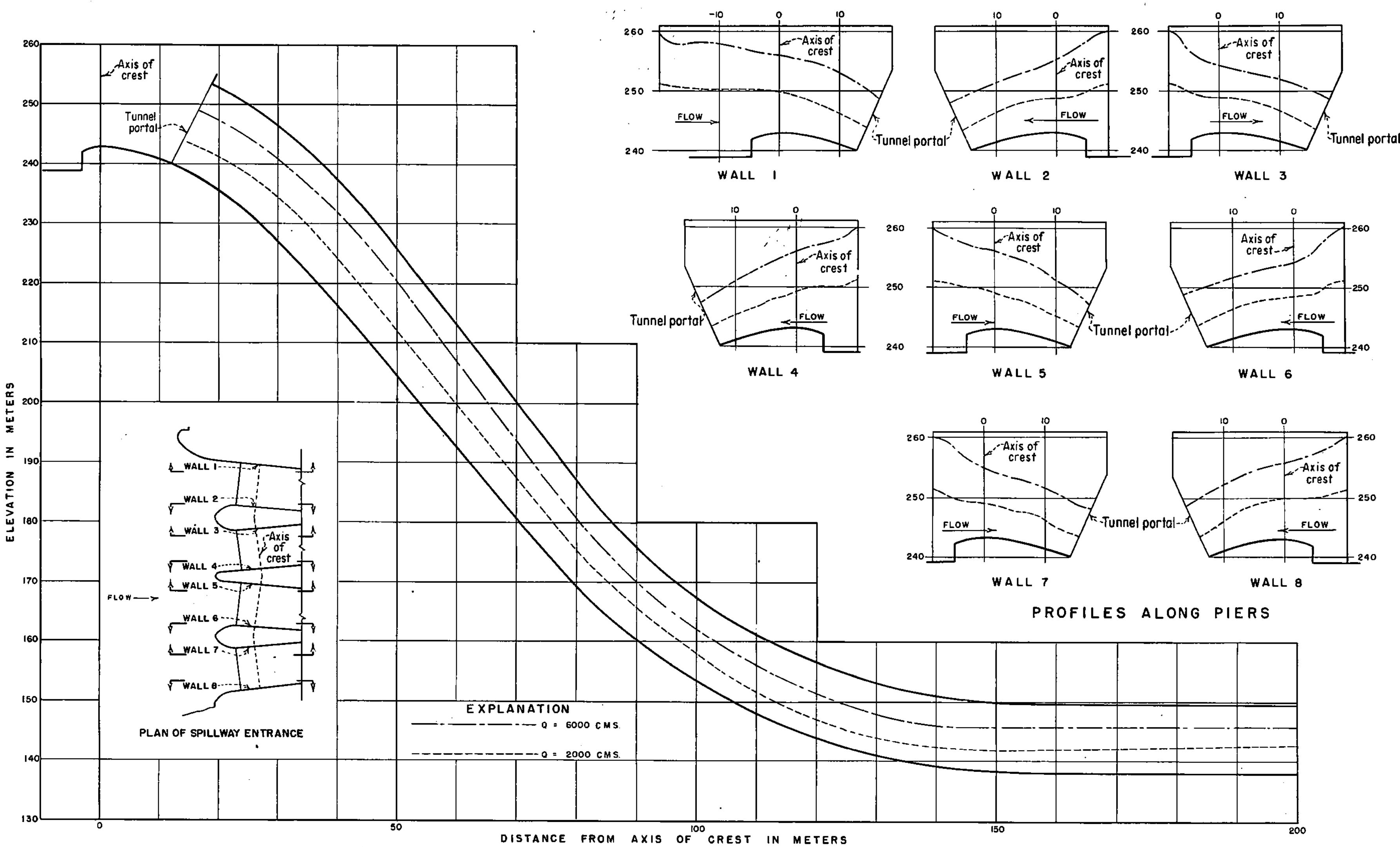
A. Flow entering spillway.

B. Flow leaving buckets.

D. Flow leaving buckets.

C. Flow in spillway tunnels.

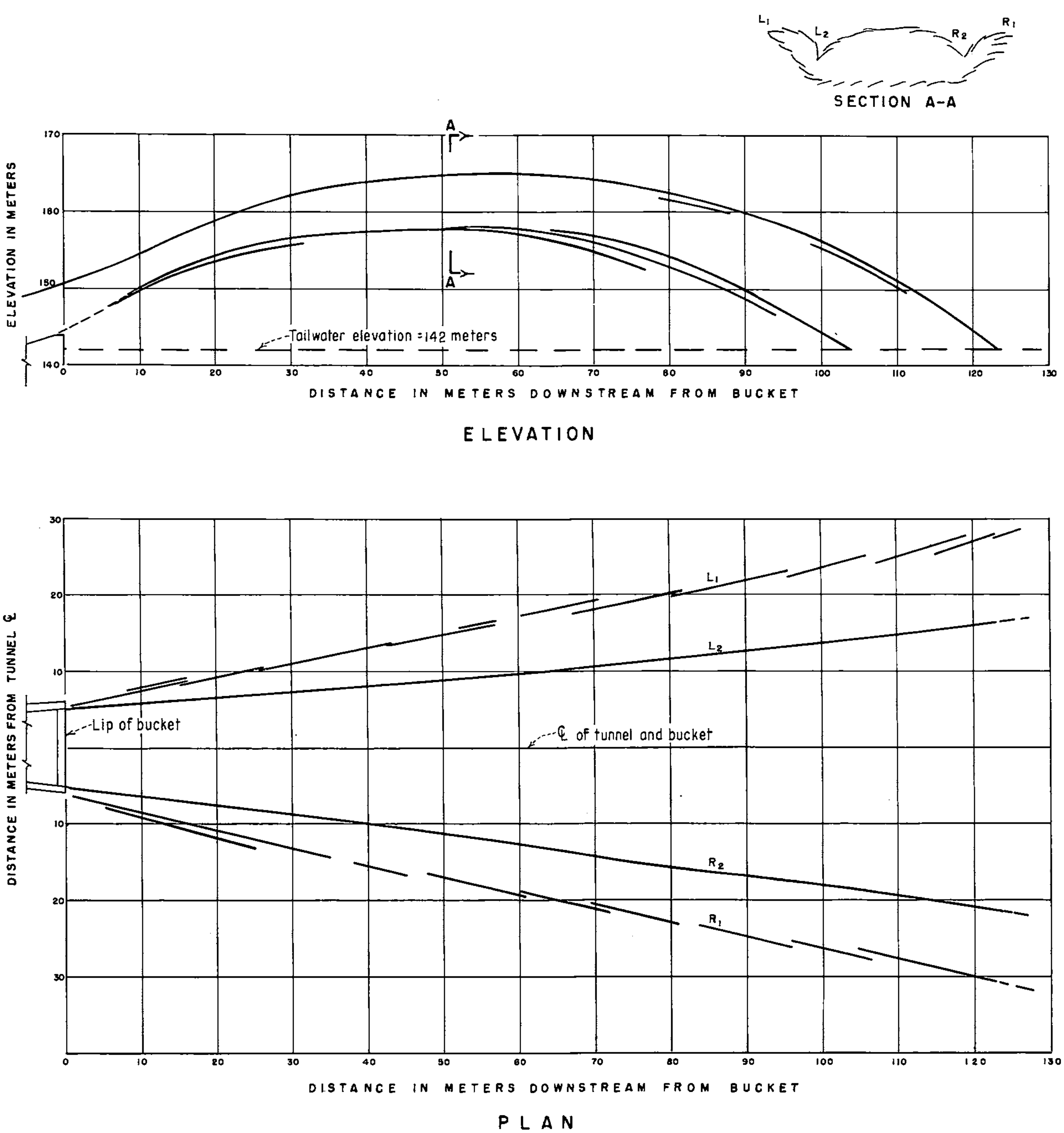
YANHEE DAM SPILLWAY
Flow Conditions for Recommended Spillway
Discharge = 2000 CMS
Gates Clear of Water Surface
1:77.37 Scale Model



YANHEE DAM SPILLWAY
WATER SURFACE PROFILE
1:77.37 SCALE MODEL

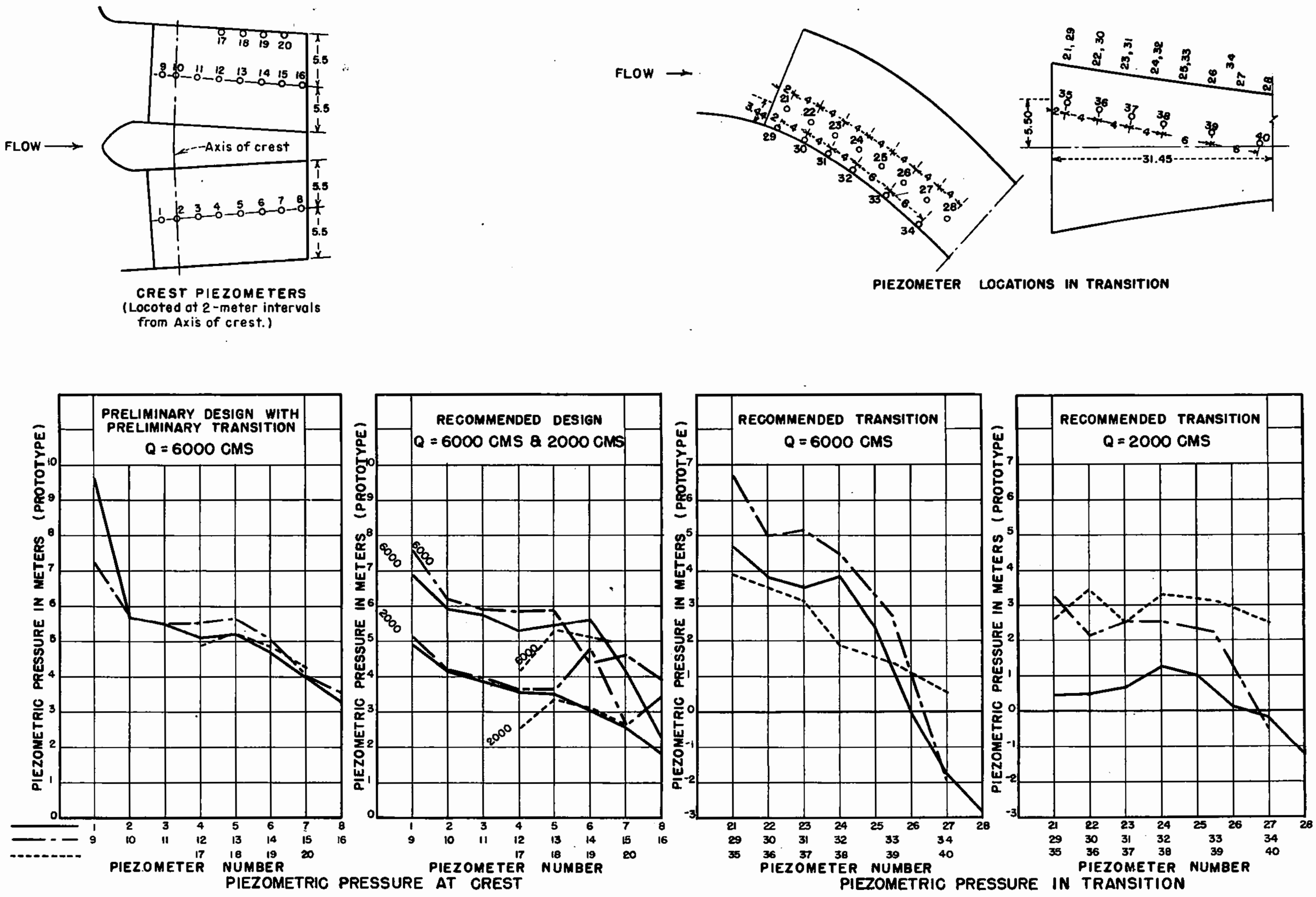
FIGURE 22
REPORT HYD 428

FIGURE 23
REPORT HYD. 428



YANHEE DAM SPILLWAY
TRAJECTORY OF SPILLWAY FLOW DOWNSTREAM FROM BUCKETS
DISCHARGE = 6000 CMS.
1:77.37 SCALE MODEL

539



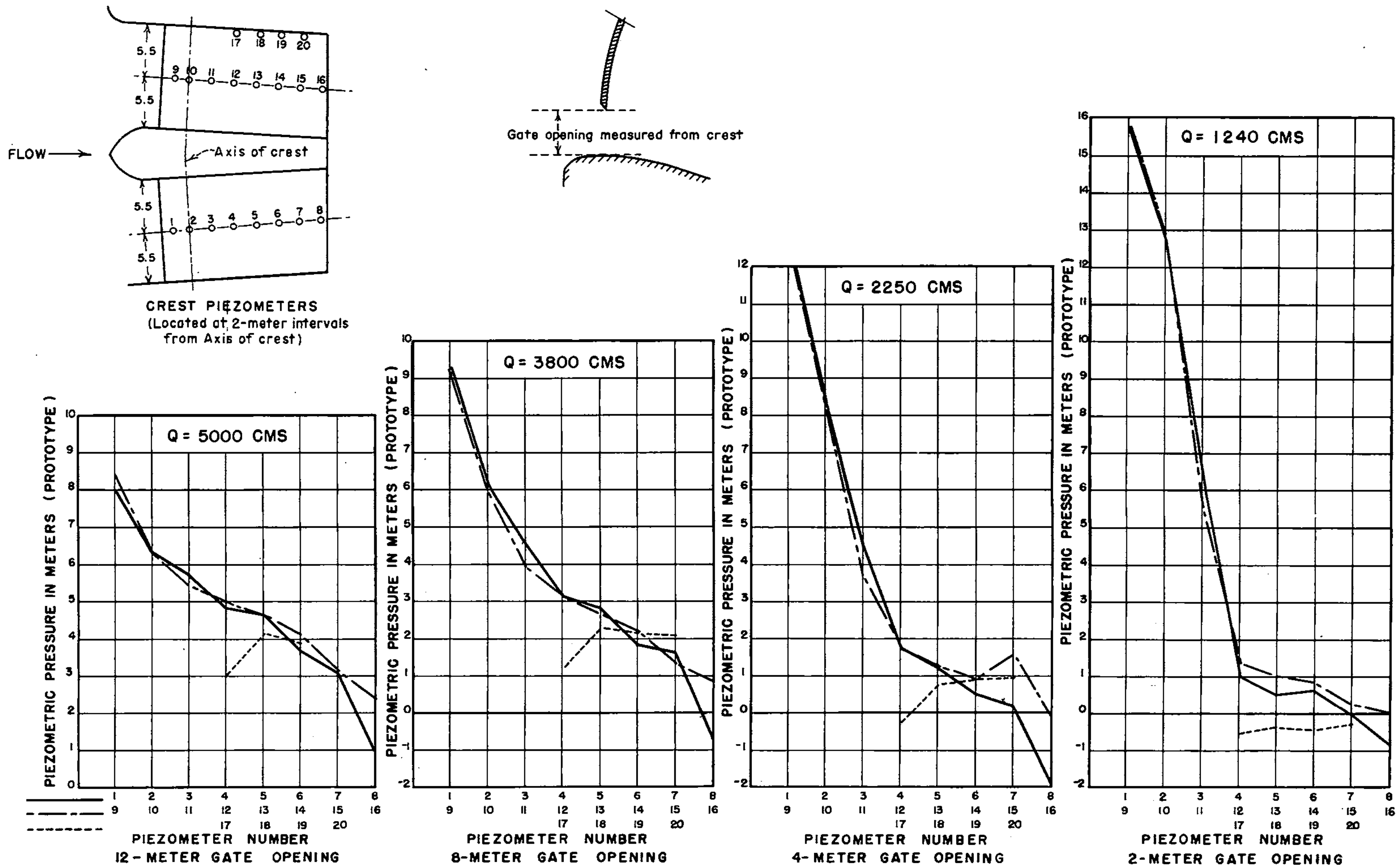
YANHEE DAM SPILLWAY

PRESSURES ON CREST AND TRANSITION

FREE FLOW

1:77.37 SCALE MODEL

FIGURE 24
REPORT HYD. 428

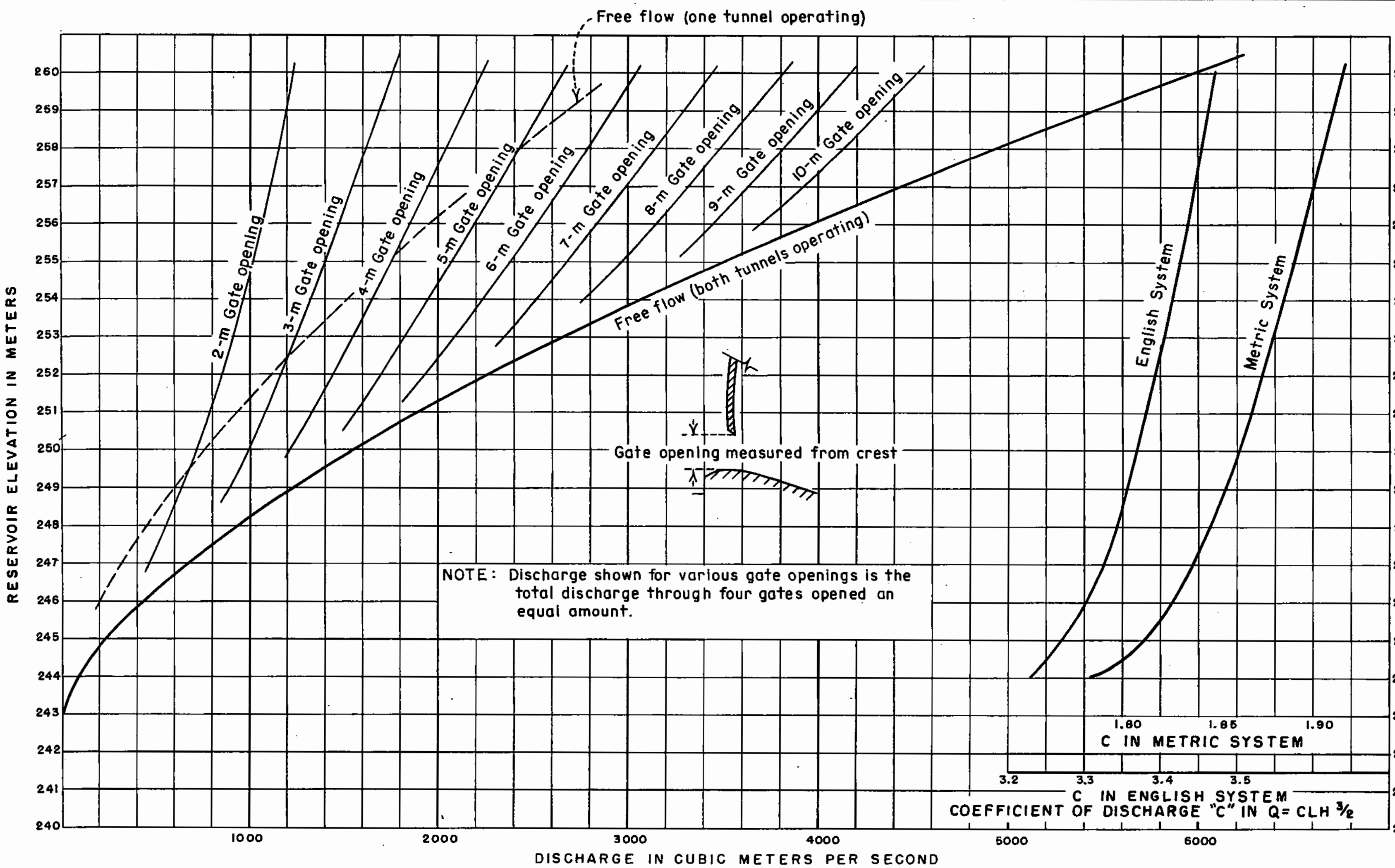


YANHEE DAM SPILLWAY
PRESSURES ON CREST FOR PARTIAL GATE OPENINGS
1:77.37 SCALE MODEL

A. Preliminary bucket design. Sand elevation before test was 136 m.

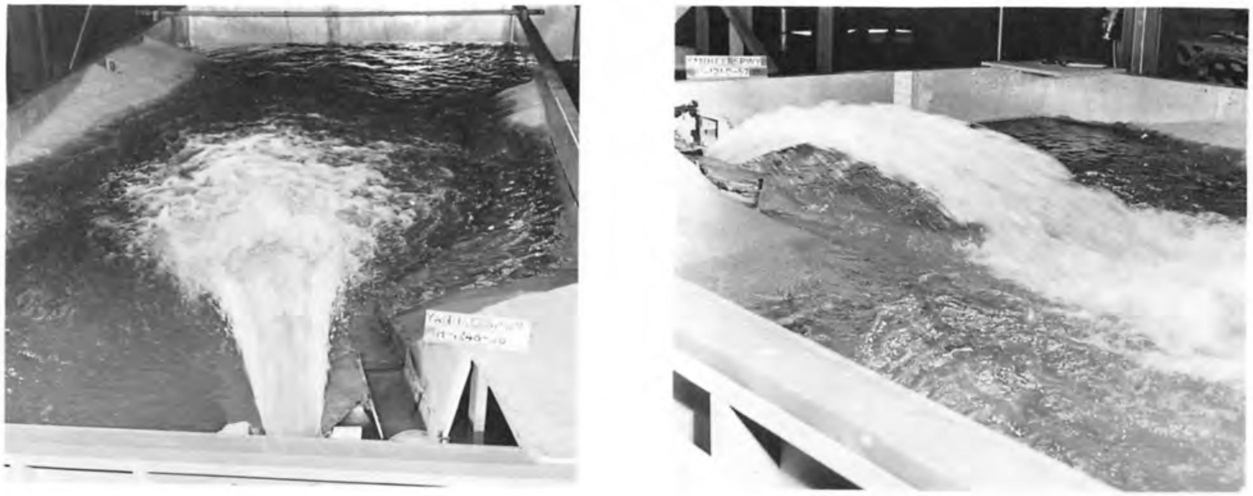
B. Recommended bucket design. Sand elevation before test was 128 m.

YANHEE DAMP SPILLWAY
Erosion of Riverbed
Model Operated 1-1/2 Hours at Maximum Discharge of 6000 CMS
1:77.37 Scale Model

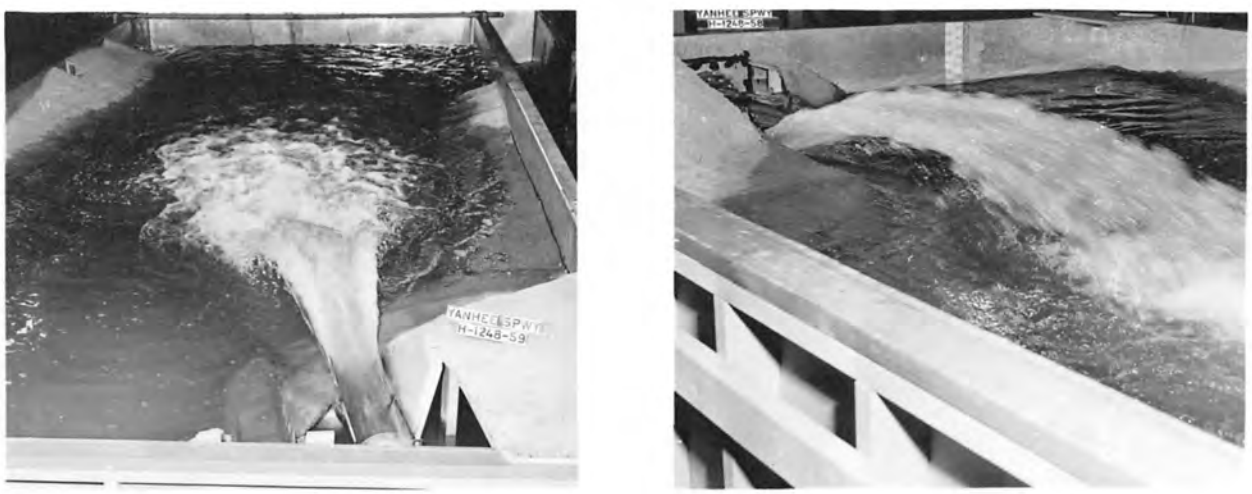


YANHEE DAM SPILLWAY
HEAD-DISCHARGE CURVES
1:77.37 SCALE MODEL

FIGURE 27
REPORT HYD. 428

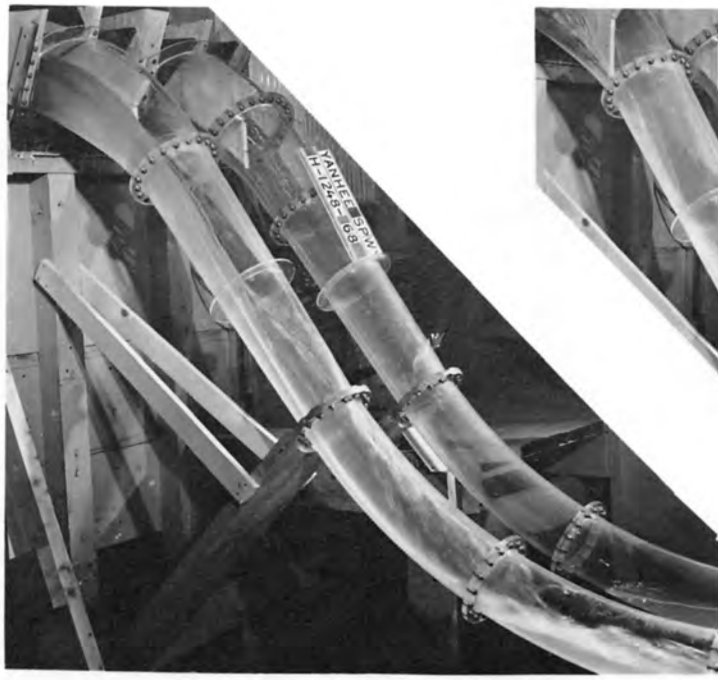


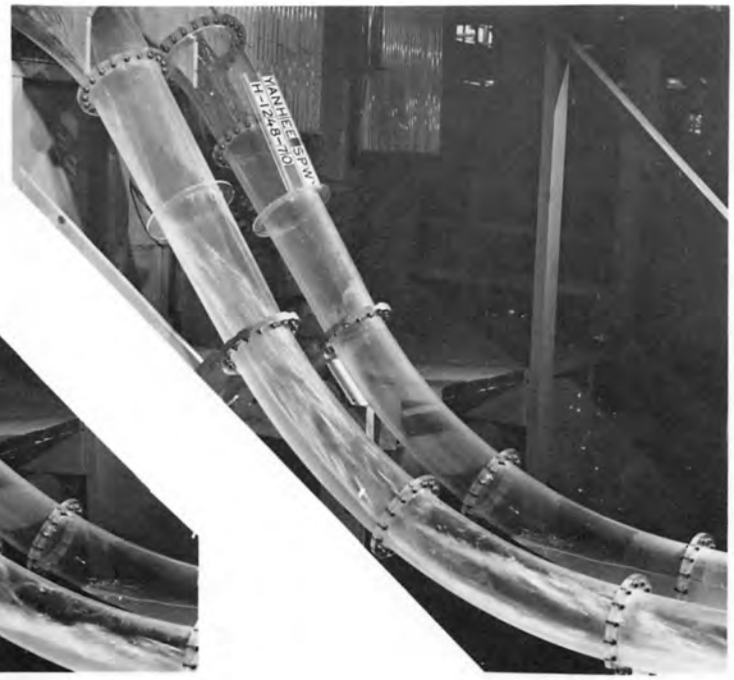
A. Flow leaving left bucket.



B. Flow leaving right bucket.

YANHEE DAM SPILLWAY
Flow from Recommended Buckets, One Tunnel Operating
Discharge = 3000 CMS at Maximum Reservoir Elevation
Gates Clear of Water Surface
1:77.37 Scale Model





Flow in tunnel.

Flow in bucket

A. Extreme right gate open - other gates closed.

B. Second from right gate open - other gates closed.

YANHEE DAM SPILLWAY
Distribution of Flow With One Gate Open
Recommended Design. Discharge = 1500 CMS
1:77.37 Scale Model