UNITED PROVINCES, INDIA

PUBLIC WORKS DEPARTMENT

IRRIGATION BRANCH

Report on

HYDRAULIC MODEL TESTS FOR RIHAND DAM

Prepared by

Civil Engineering Section
Colorado Agricultural Experiment Station
Fort Collins, Colorado

For

International Engineering Company, Inc.
74 New Montgomery Street
San Francisco, California

April, 1950

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COLORADO AGRICULTURAL EXPERIMENT STATION

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FOREWORD

The studies described in this report were made during the period from August 1949 to February 1950 in the Hydraulic Laboratory of Colorado Agricultural and Mechanical College, Fort Collins, Colorado. Construction and testing of the hydraulic model of Rihand Dam was authorized in a contract between the Colorado Agricultural Research Foundation of the Colorado A & M College acting through the Civil Engineering Section of the Experiment Station, and the International Engineering Company, Inc., San Francisco, California.

The International Engineering Company, Inc., is responsible for the design of the structure, and their Chief Engineer, Mr. D. J. Bleifuss together with Mr. N. L. Hinkson observed the model in operation and discussed the results with laboratory staff members. Throughout construction and testing of the model, consultations with and inspections by Mr. W. A. English, a representative of the company, were maintained at regular intervals.

Professor T. H. Evans is Dean of Engineering and Chairman of the Engineering Division of the Experiment Station. Dr. D. F. Peterson is Chief of the Civil Engineering Section of the Experiment Station.

Laboratory staff engineers who contributed to the model studies were Mr. A. R. Robinson, in charge of the design office and the construction of the model, Mr. D. Q. Matejka, in charge of the testing of the model, and Mr. C. H. Zee, assistant to Mr. Matejka. Professor S. D. Resnick supervised the compilation of the material for the report.

Mr. A. J. Peterka was consultant to the laboratory on the testing program. He was assisted by Mr. W. E. Wagner. The entire program was under the direct supervision of Dr. Maurice L. Albertson.

The report was written by Professor Resnick and Dr. Albertson, and later rearranged and edited by the International Engineering Company, Inc.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

Chapter I

INTRODUCTION

General

Rihand Dam is planned by the United Provinces, India on the Rihand River about 29 miles upstream from the confluence of the Rihand and Sone rivers. The site, at an elevation of about 700 feet above sea level, has a tributary watershed of 5,148 square miles. The reservoir created will have a storage capacity of 8.8 million acre-feet at Elevation 880.0. The project will produce hydroelectric power, provide storage of water for irrigation, and afford a considerable degree of flood control on the Rihand River below the dam. Partial flood control will also be effected on the Sone River. General relative locations are shown on Fig. 1.

Need for Hydraulic Model Studies

The Public Works Department, United Provinces, procured the services of the International Engineering Company, Inc., San Francisco, California to design the dam, powerhouse and appurtenant works. Dr. John L. Savage of Denver, Colorado, was retained by the United Provinces as Consulting Engineer for the Rihand River development project. It was decided by the Public Works Department, United Provinces, the designers, and the consulting engineer, that hydraulic model studies would be desirable in order to check some of the important design features of the project. Accordingly, the Colorado Agricultural Research Foundation of Colorado A & M College was engaged by contract with the designers in August, 1949, to perform the model testing work and submit a report on its findings. Model work has been conducted at the hydraulic laboratory of Colorado A & M College at Fort Collins, Colorado.

Scope of Investigations

It was mutually agreed among the agencies concerned that a report on model tests should be prepared for submittal to the Chief Engineer of the Public Works Department, Irrigation Branch, United Provinces, India. The report was to cover pertinent details of model tests with necessary sketches, pictures, interpretations of data, and recommendations.

A 1:72 scale model of the Rihand Dam and the immediate surrounding area was built and tested over a wide range of possible operating conditions. The tests were limited to the design originated by the International Engineering Company, Inc., for this structure. Modifications recommended as a result of these tests were not investigated by model tests but are believed to be necessary for best hydraulic performance.

Prototype Structure

The Rihand project comprises a dam with overfall spillway, outlet works, and powerhouse with appurtenant works. Drawings furnished by the designers, giving structural details of the project, represent the prototype of the investigations undertaken in this model study program.

Dam: The dam will be located in a comparatively narrow portion of the river valley - see Fig. 2. It will be of concrete gravity type with a maximum height of about 296 feet above the foundation and a length at roadway level of 3,004 feet. The alignment is straight across the river channel with a portion of each abutment curved to fit the topography. Due to foundation conditions, the east abutment ends in an earth embankment about 138 feet long. Figures 3 and 4 show general layout and typical sections.

Spillway: The ogee-type spillway has a gross length of 664 feet and is designed to pass a maximum flood of 440,000 cfs under a net head of 36 feet. Flow over the spillway will be controlled by 14 tainter gates each 40 feet wide and 28 feet high. Top of the closed gates is at Elevation 880.0. As designed, the energy of the overflowing water will be dissipated by a hydraulic jump stilling pool. The stilling basin consists of a 325-foot long sloping apron without baffles, but with an end sill 5 feet high. Vertical and parallel training walls extend to the end of the apron and are continuations of the spillway training walls. The spillway face is joined to the apron by a simple bucket having a radius of 120 feet. Figures 5, 6, 7 and 8 show various details of the spillway.

Regulating Works: Irrigation and other regulatory flow releases from the reservoir, as may be required in addition to turbine discharge, will be provided by two sluices located in the spillway section at Elevation 710.0. Details are shown on Fig. 9 and discharge rating curve on Fig. 10.

Power Plant: The power plant is located to the east of the spillway immediately downstream of the dam. A total of six hydro-turbine generators will be installed, each of 43,750 kva capacity. Penstocks, controlled by fixed-wheel gates at the intake, extend through the dam to serve the turbines. Average total discharge from six turbines will be about 15,000 cfs. The tailrace is separated from the stilling basin by the right training wall.

Model Size and Scale

Before constructing the model of Rihand Dam, the various hydraulic features were studied and all points of potential difficulty were noted. With this information in mind, the scale of the model was determined, governed also by the following three factors:

1. Data necessary to evaluate the performance of the structure.

2. Degree of accuracy required of the data.

3. Laboratory facilities, such as capacity of pumps and available floor space.

When all factors were considered, it was found that a model scale of 1:72 would fulfill requirements. Rihand Dam has several independent hydraulic elements. In order to determine the effect of each element on other elements, an assembled or "complete" model was necessary.

The model was of sufficient size so that calibration data, pressure distribution, and the general flow pattern in most features could be obtained without the need for larger separate models. A model scale smaller than 1:72 would not have permitted accurate measurement of spillway and sluice discharges.

Chapter II

MODEL CONSTRUCTION AND TESTING

Design and Construction

General Layout: As may be seen in Fig. 11 the complete model was contained in a head box upstream and a tail box downstream from the dam. The upstream face of the tail box formed the upstream face of the dam. The entire model occupied a floor space of 864 square feet. Care was taken to make the head and tail boxes sufficiently large to include the necessary topography upstream and downstream. The area was sufficient to insure reasonable reproduction of prototype flow conditions upstream and downstream from the dam.

In order to provide a sufficient depth of bed material downstream from the stilling basin, the model apron was built one foot above the tail box floor. This depth of bed material made it possible to measure seventy feet of prototype erosion without exposing the tail box floor. It also provided a margin of safety in the event that it became necessary to lower the apron. The floor of the head box was constructed considerably higher than that of the tail box (See Fig. 11) to effect economy in construction.

For the Rihand model, gravitational forces were considered to predominate; thus the model was designed according to Froude's law, the Reynolds number being considered relatively unimportant.

Head Box and Tail Box: The head box was made 24 feet wide by 8 feet long and 4 feet deep as shown on Fig. 11. The 8-foot dimension was the minimum possible to provide smooth uniform flow entering the spillway. The 24-foot dimension was governed by the economy of a rectangular layout, and the need for a 24-foot width in the lower channel. A 4-foot depth was required for the dam and necessary freeboard. A 6-inch thick rock baffle was placed across the entire width of the head box to dissipate the energy of the water jet from the 14-inch supply pipe. Topography in the head box was formed in accordance with contours shown on Fig. 11 and 3.

The tail box was made 28 feet long, 24 feet wide and 2.67 feet deep as shown on Fig. 11. At the downstream end of the tail box, a cut-off wall was constructed to approximately Elevation 626, which was the elevation of the river bed at this section. This wall also formed the upstream side of the sand trap used to trap bed material carried down the channel. A tail gate was built downstream from the sand trap to control the tailwater elevation. A chute below the tail gate guided the water into the return channel.

Various details of the model construction are shown on Plates 1 to 14 inclusive.

Overfall Spillway: The face of the spillway was constructed of half-inch plywood fastened to wooden A-frames. Spillway crest and bucket were made of masonite treated with linseed oil before being curved and fitted into place.

A single row of piezometers was provided in a template located on the centerline of bay No. 3 (See Plate 6). Piezometer tubes were carried under the crest and out the side of the spillway to a manometer board. Spillway piers made of Honduras mahogany and treated with waterproofing material were fitted to the crest (See Plate 13). Tainter gates were constructed of sheet metal and hinged to the piers. The spillway training walls were made of half-inch plywood and fastened to the A-frames.

Air Step: A small brass angle fastened to the spillway formed the downstream face of the air step. The space between this brass strip and the point of tangency was filled with modeling clay carefully formed and smoothed into proper shape. This construction would allow easy modification of the air step in case it was found necessary.

Sluices: Since the purpose of the sluices in the model was to study outlet conditions, no attempt was made to duplicate the intake end exactly. The relatively small size of sluices in this model made it impossible to obtain useful information on capacity or entrance pressures. Accordingly, the model sluice entrances and barrels were designed primarily to pass the discharges computed by the designers. The outlet ends were constructed geometrically similar to the prototype in order to insure hydraulic similitude between model and prototype sluice discharges. Model sluice barrels were made slightly over size and were equipped with gate valves for regulation of discharges.

The arrangement and outline of prototype sluices is shown on Fig. 5 and 9. In the model, sluice barrels were made of 1-1/2-inch diameter copper pipe. A standard floor flange threaded to the upstream end of each pipe served as a bell-mouthed entrance. The flange also provided a simple means of fastening to the upstream face of the dam. A 1-1/2-inch gate valve was installed near the upstream end of each sluice. The outlet ends of the sluices were made of sheet metal to the shape shown on Fig. 9. The "eyebrow" above each sluice outlet shown on this figure was also made of sheet metal.

Stilling Basin: As designed, the stilling basin is 325 feet long measured from the end of the bucket and 664 feet wide (See Fig. 8). From Elevation 627.5 at the bucket, the apron slopes 1 on 13 down to Elevation 604.0 at the end sill. The end sill is 5 feet high with an upstream slope of 1 on 1 and a top width of 5 feet. The model apron was constructed of plywood over a wooden frame supported by the tail box floor. The training walls, with crests at Elevation 679.0 extending to the end of the apron, were also made of plywood over a suitable wooden frame. The end sill of the model was made of mahogany and screwed to the apron. A cut-off wall extended vertically downward from the apron to the tail box floor to prevent bed material from working back under the apron.

Powerhouse: The powerhouse was constructed of 2-inch lumber frame work covered with half-inch plywood. Plate 13 shows a portion of the partially completed powerhouse adjacent to the training wall. To simulate turbine discharge, three 2-inch pipes were connected to the powerhouse from the head box. The pipes were provided with gate valves for control of discharge. Since no turbines were constructed in the model, energy dissipation was effected by a baffle constructed in the powerhouse. The baffle was modified by trial to provide a flow in the model tailrace at a velocity corresponding to that expected in the prototype.

Equipment and Procedure for Testing

In order to clarify the use of laboratory equipment and the procedures followed for making accurate measurements of discharge, pressure, water surface elevation and erosion, a discussion of the testing equipment and its use is given below.

Measurement of Discharge: Water was supplied to the model through a 14-inch pipe from a 20-hp propeller-type pump. Two calibrated orifice plates, 10-1/2-inch diameter for large flows and 5-inch diameter for small flows, were used to determine the discharge.

Measurement of Pressure: Pressure measurements were made on the crest of the model spillway and immediately downstream from it. To accomplish this, 10 piezometers were placed flush with the surface of the crest in the center of bay No. 3 ranging from the upstream lip of the crest to a point at Elevation 800.5 downstream from the air step. Each piezometer was directly connected by a separate flexible tube to one of the glass tubes in the manometer bank. Thus the pressure at each piezometer location could be quickly compared with that at any other piezometer location. Each piezometer was primed in the standard manner with a solution of 90% water, 5% Aerosol and 5% fluorescein. The fluorescein gave the water a color which made the meniscus readily visible. Aerosol was added to destroy surface tension and avoid capillary rise in the 1/8-inch glass manometer tubes.

Measurement of Air Entrainment: In tests made by the Bureau of Reclamation, it has been demonstrated that cavitation pitting is materially reduced by the presence of air in the water at the boundary surface. For this reason attempts have been made recently in the design of hydraulic structures to incorporate devices for introducing air. In the case of Rihand Dam, an air step six inches high is placed on the spillway crest immediately downstream from the piers. Air intakes are located in the downstream face of each spillway pier.

Conduits lead to headers embedded in the spillway beneath the air step. From these headers 10-inch diameter pipes at 5 feet on centers open up onto the spillway face just downstream from the step. Some additional air under the nappe will also be introduced at a point immediately downstream from the spillway piers. The air thus supplied should tend to reduce any erosive tendencies that may exist.

The amount of air entrained will depend on several factors, such as:

1. Pressure under the nappe.
2. Roughness of the underside of nappe, which determines to a large extent the ability of moving water to drag or "pump" the air.
3. Size and shape of the void space beneath the nappe.
4. Velocity of water in the nappe.

It is known that the model does not reproduce the air entrainment conditions of the prototype. However the model may be used as a qualitative indicator of what may be expected in the prototype.

To measure the air entrained in the model, it was necessary first to seal off the underside of the nappe downstream from the piers. Next, it was necessary to introduce air under controlled conditions. This was accomplished by using an air compressor to supply air through a vent immediately downstream from the air step. The amount of air introduced was measured by an orifice meter placed in the line between the compressor and the vent. The orifice was of standard construction and the pressures immediately upstream and downstream from the orifice plate were measured by a water manometer.

<u>Measurement of Water Surface Elevation</u>: Water surface elevations were measured by Lory-type adjustable gages mounted at three locations as shown on Fig. 11. The gages read to one thousandth of a foot model scale. The headwater hook gage was mounted in a 4-inch plastic stilling well to facilitate reading. Tailwater and tailrace point gages were unprotected.

A quick-setting float gage was used to set the headwater elevation. Although this gage could not be used for final readings, it was extremely sensitive and greatly facilitated the process of setting a new headwater elevation. The final and more refined adjustments were made using the Lory gage.

Heights of surges in the tailrace and of waves downstream from the stilling basin were measured by point gages. The gages were set to the maximum and minimum water surface elevations occuring over periods of several minutes.

Measurements of Erosion: Before each erosion test, the bed of the model was shaped as nearly as possible to that which presently exists at the damsite (See Fig. 3). The model was operated for 60 minutes during each test, after which the flow was stopped and the model carefully drained. Erosion contours were then determined by means of a "water level" consisting of a reservoir connected by a flexible tube to a glass manometer graduated in prototype feet. The movable reservoir was set near and above the eroded bed and the manometer base placed on some known datum, such as the end sill. Next, the scale on the manometer was adjusted to read directly the elevation of the datum. The manometer then indicated the prototype elevation of any point upon which it was placed. Contours were determined by moving the manometer over the river bed. To facilitate photographing the extent of erosion, each contour was outlined with heavy white string.

Test Results for Overfall Spillway

The overfall spillway was tested with discharges for the probable range of head water elevations. Flow patterns throughout the structure were carefully noted, and spillway capacity determined with gates fully or partially raised. In conjunction with the operation of the spillway, pressures on the crest were measured.

The model tests were studied and analyzed in an attempt to predict prototype performance where experience has shown that model and prototype operation differ sufficiently to require interpretation. An attempt has also been made to analyze the operation and indicate, where possible, why the structure performed as it did.

Flow Patterns: Generally, the spillway performed reasonably well. Proper discharge was obtained with flow patterns that were satisfactory. Flow approaching the spillway was smooth and uniform, and no difficulties were evident in the approach area.

For low and medium discharges the flow condition near the spillway piers was satisfactory. However, as the flow was increased, the drawdown at the piers increased until, at a discharge of about 450,000 cfs, the surface became very irregular. A depression was formed around the pier noses as shown on Plates 15 and 16. Except at the end piers, this was of no particular concern. Since the piers project 7-1/2 feet upstream from the face of the adjoining non-overflow section, there was considerable flow of water around end piers into the spillway. Thus, much of the flow entering the end bays was forced to turn on a very small radius and accelerate at the same time. The result was a contraction in the flow and a considerable depression of the water surface around the end piers as shown on Fig. 12. Although the reduction in overall capacity caused by the depressed water surface was minor, there was a material reduction for the two end bays alone. Furthermore, after the nappe contracted as described, it struck the training wall just below the downstream end of the spillway piers, spilling over as shown on Plate 17. The tests demonstrated that changes in design of the end piers are desirable.

Profiles of the flow over the crest are shown on Fig. 12 and 13 and Plate 15. For the maximum headwater, Elevation 888, it is apparent that the gate pins are safely above the maximum height of the water surface. It is not expected that entrainment of air at this point in the prototype will be sufficient to raise the water surface above the profile shown for the model.

The flow on the spillway face with spillway operating alone was satisfactory except for the effect of the end piers previously described. With sluices and spillway both operating, the "eyebrows" located just above the sluice outlets effectively prevented interference by spillway flow with sluice discharge. Only insignificant disturbances in the flow pattern on the spillway face were created.

In leaving the spillway face, the flow entered the bucket satisfactorily. The bucket radius appeared sufficiently large to conduct the flow smoothly into the stilling basin.

Pressure Distribution: Spillway pressures are of importance. Sufficiently low pressures would indicate a tendency for cavitation in the prototype, while unusually high pressures would indicate an inefficient crest. As shown on Fig. 12, 13 and 14 pressures were found to be near or greater than atmospheric pressure for every condition tested. The crest as designed is therefore considered satisfactory from the standpoint of pressure distribution.

Spillway Capacity: The capacity of the overfall spillway was determined for a range of headwater from Elevation 852.0 (top of crest) to Elevation 888.0. Free discharges over the crest and discharges through partially raised gates were measured. The gates were opened uniformly at all times. With reservoir at Elevation 880.0 and tainter gates fully raised, the spillway capacity was found to be 304,000 cfs. With reservoir at maximum flood stage, Elevation 888.0, the discharge was found to be 454,000 cfs.

Flow profiles indicate that it is necessary to open the spillway gates only 24.9 feet to obtain free flow with headwater at maximum Elevation 888.0. Headwater-discharge curves for free flow and for flows through partial gate openings of 2, 4, 8, and 16 feet are shown on Fig. 15.

Fig. 16 shows curves of discharge coefficient versus reservoir elevation for free flow and for flow with gates partially raised. The discharge coefficient for free flow was obtained from the equation

$$Q = CLH^{3/2}$$

and for flow through partial gate openings from the equation

$$Q = CLb(2gH)^{1/2}$$

where Q is the discharge in cubic feet per second, C is the coefficient of discharge, L is the length of spillway crest in feet, H is the difference in elevation between the reservoir water surface and the spillway crest, and b is the gate opening.

In determining total spillway capacity, sluice and crest discharges were added together.

Test Results for Stilling Basin

At maximum discharge the hydraulic jump in the stilling basin extends upstream to the bucket, making the effective length of the apron approximately four times the pool depth. The approximate maximum flow per unit width of apron is 680 cfs per lineal foot.

Tests of stilling basin performance were run over a range of combined discharges, from the overfall spillway and sluices, of 9,230 cfs to 467,400 cfs. Erosion tests were made over a range of combined discharges from the overfall spillway, sluices, and powerhouse of 28,400 cfs to 477,000 cfs to determine the erosive tendencies at the end of apron and training walls and in the channel downstream. Surges and wave heights were measured in the powerhouse tailrace and in the lower river channel. Water surface profiles were photographed in the stilling basin to show the general character of the water surface. Tests were made in which tailwater was raised or lowered in relation to its "normal" elevation. "Normal" tailwater elevation for a certain discharge is that given by the tailwater rating curve shown on Fig. 10. Resulting hydraulic jump profiles were studied in order to determine if apron elevations as shown on the drawings would be satisfactory. Tests and measurements mentioned in this paragraph are discussed in detail hereafter.

General Performance: The stilling basin, in general, performed satisfactorily over the entire range of discharges. The required flows were passed without causing serious erosion damage, either to the structure itself or to the channel downstream.

Table I summarizes the results of tests made to determine the position of the jump for various discharges and depths of tailwater. It is apparent from these tests that the position of the jump is very sensitive to variations in tailwater depths. Although deviations from normal tailwater used in the model tests may seem excessive, it is thought that variations of similar magnitude may appear in the prototype for the following reasons:

1. Unknown factors may cause the tailwater curve to differ from that shown on Fig. 10.

2. When spillway discharge is increased rapidly, there is a time lag before tailwater can rise to the normal level. During such periods an actual reduction below normal tailwater depth occurs.

3. Even though erosion downstream from the structure is slight, some degradation of the stream bed may occur. Clear water discharged from the reservoir will pick up silt in the downstream channel. Thus over a period of years the tailwater level could be lowered considerably.

Tests with tailwater above normal, approximated the conditions which would prevail with the entire apron lowered the distance that the tailwater was raised. In these tests the jump formed closer to the spillway face, appeared to be more stable, and the stilling pool surface was noticeably smoother. Although the jump was drowned slightly for some discharges, the loss of efficiency in the jump was negligible, and fewer waves appeared in the downstream channel. Also, with the jump in this upstream position, the apron appeared longer than necessary.

It is believed, although no confirming tests were made, that if the apron were lowered 10 feet, the apron length could be reduced at least 50 feet. At the same time the jump should become more stable with respect to changes in tailwater elevation, and a quieter water surface should result in the downstream channel. It is believed also, that further tests would show that the entire apron need not be lowered 10 feet. It is possible that the downstream end could be held at Elevation 604.0, and the slope flattened to provide the additional depth at the upper end.

Certain water-surface profiles were recorded as shown on Fig. 17 and 18. Flow patterns and water-surface profiles in the stilling basin were photographed and are shown on Plates 17 to 27 inclusive. An inspection of the profiles in the stilling basin shows that for 477,000 cfs, the maximum discharge used in the tests, and with tailwater at Elevation 679.7 (about 1.3 ft below normal), the training walls were overtopped by wave action, (See Plate 17). The inflow, however, was so slight that it had no measurable effect on the efficiency of the stilling basin. When the tailwater was raised to Elevation 689.7, about 8.7 feet above normal level for this discharge, there was a head of 10.7 feet over the top of the walls. Even this amount of inflow at the sides of the basin was found to reduce the efficiency of the basin but little. Since maximum floods are rare, it seems reasonable to assume that the height of the training walls could be somewhat reduced without detrimental effects.

The sluices were tested only to determine the effect of the outflow on the performance of the stilling basin. For spillway discharges greater than 69,500 cfs, the effect of sluice operation on stilling basin performance was neglegible. However, the effects of sluice flow could be detected in the erosion tests as will be discussed later.

For total discharges of 69,500 cfs and less (See Plate 23) the effect of sluice operation was manifested by increased disturbances in the stilling pool flow pattern. The water surface profile became irregular and the force of the concentrated sluice jets caused two indentations at the toe of the jump. This resulted in some instability of the hydraulic jump.

Waves and Surges: Waves and surges in the lower channel and in the powerhouse tailrace were studied. These disturbances originated in the stilling basin as a result of the action of the hydraulic jump.

The height of the maximum surge in the tailrace of the model was determined by means of a point gage located as shown in Fig. 11. The elevations of the highest crest and lowest trough in open water were measured, the difference between the two elevations being considered as the height of the surge. These measurements are summarized in Table II for various discharges. A total of 6 tests were made in which the combined flow from the spillway, sluices, and powerhouse was varied from 28,500 cfs to 477,000 cfs. Due to fluctuations in surge heights, it was difficult to measure instantaneous water surface elevations closer than about 0.01 feet in the model, which is 0.72 feet in the prototype. The surges in the tailrace were considerably less than wave heights in the downstream channel. This is due principally to the dampening effect of the right training wall.

It was found that the height of the surge increased with discharge, and that for the same discharge the surge decreased with a decrease in tailwater depth. The waves and surges, although not excessive even under conditions of maximum flow, could conceivably cause extensive erosion along downstream banks. As discussed earlier, it is believed that with modification of the apron, a smoother water surface in the downstream channel could be obtained.

Erosion: Exact information as to the character of the river bed immediately downstream from the stilling basin is not available. Data at hand indicates that rock is exposed or near the surface.

In order to establish erosive tendencies, an average prototype size of 0.5 inch bed material was assumed. To obtain dynamic similarity, the bed material in the model consisted of loose sand with a 0.025 inch mean grain size. The determination of size was based upon the work of Krumbein (1), Rouse (2), and Doddiah (3) who established the principle that the fall velocity of a particle reflects its susceptibility to erosion as either bed load or suspended load. Following this principle, the ratio

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1. Krumbein, W. C. — Settling velocities and flume behavior of non-spherical particles. American Geophysical Union Transactions, 1942: 621-33.

2. Rouse, Hunter — Criteria for similarity in the transportation of sediment. Proceedings of Iowa Hydraulics Conference (1939). State University of Iowa Studies in Engineering. Bulletin No. 20.

3. Doddiah, D. — Comparison of scour caused by hollow and solid jets of water. Thesis, in partial fulfillment for the Degree of Master of Science, Dec. 1949, Colorado Agricultural and Mechanical College.

w/v was kept a constant from model to prototype, where w is the fall velocity of the bed material and v is the characteristic velocity of the flow. On a strictly geometric scale relationship, in contrast to dynamic similitude, the model bed material represented loose rocks in the prototype having an average size of 1.8 inches.

The river bed of the model was movable below Elevation 720. Above this elevation the river banks were constructed of non-erodible concrete mortar placed on metal lath formed to approximate contours of the prototype.

In making an erosion test, the erodible bed was molded to the proper shape and the lower river channel was filled slowly to prevent premature movement of the bed material. When the tailwater reached an elevation sufficient to produce the hydraulic jump for the discharge being tested, flow over the spillway was started. The discharge and tailwater were then set, and tests allowed to run for one hour. At the end of this time the model was carefully drained. Levels were taken to establish the contours which were then outlined with white string and photographed. Tests were run for one-hour durations because it was found that no appreciable change occured in the erosion pattern after this time. Furthermore, by continuing each test for a uniform length of time, a basis for comparing tests was established.

Erosion at the end of the apron was moderate regardless of the discharge or tailwater condition tested. Even for a discharge of 477,000 cfs, there was no undermining of the apron or training walls. Erosion tests are summarized in Table III.

The deepest erosion occured at the end of each spillway training wall and was the result of a subsurface eddy. Flow leaving the end sill was directed upward, inducing a cross current moving inward under the main flow. At the right wall, the river bank was sufficiently far removed to permit this eddy to develop fully. On the left side, due to the proximity of the bank, only a weak current was observed.

Generally, as may be seen from Table III, extent of erosion decreased with a decrease in discharge. With a flow of 477,000 cfs, the maximum depth of erosion occurred at the right training wall and was 32 feet below the top of the end sill (See Plates 28, 29, and 30). As stated before, due to the smaller eddy developed at the left training wall, the erosion was considerably less than at the right wall. Two additional areas of erosion developed in the channel immediately downstream from the sill and were caused by the two jets of flow from the sluices (See Plate 29). These twin scour channels were approximately 300 feet long, 100 feet wide and 15 feet below the top of the end sill at their deepest points.

With a discharge of 334,500 cfs, tailwater at Elevation 666.5, which is six feet below normal, crest gates and sluices open, and powerhouse operating as before, erosion was somewhat reduced. For the same discharge and operating conditions, but with tailwater lowered an additional five feet, erosion was changed but little. It should be pointed out that the extent of erosion has been shown by Doddiah (3) to increase with water depth to a certain maximum point and then decrease with further increase in depth. It is difficult to say on which side of the maximum point the Rihand erosion may be.

At a flow of 69,500 cfs, tailwater at Elevation 644.0, which is six feet below normal, with crest gates opened two feet and sluices and powerhouse operating, it was found that the small erosion in the downstream channel was caused chiefly by scour due to flow from the two sluices. The sluice jets set up eddies on the apron which were not counteracted by the relative small crest flows. Hence bed material was picked up by the eddies and carried back onto the apron (See Plate 26). With similar flow and operating conditions, but with a lower tailwater, it was found that the erosion in the downstream channel due to the sluice jets was somewhat increased. Generally, with a given discharge, as tailwater depth was lowered the energy dissipation on the stilling basin became less complete, and erosion downstream increased.

With only the sluices and powerhouse operating, a discharge of 28,500 cfs and tailwater about normal, total erosion was negligible. However, more material was deposited on the apron than in previous tests due to eddies set up by the flow from the sluices.

With a discharge of 390,000 cfs, tailwater about 9 feet above normal and only the right 12 spillway bays operating, the jump angled irregularly across the apron (See Plate 27). The resulting erosion was concentrated downstream from the right training wall in a scoured channel approximately 450 feet long, 440 feet wide and with a maximum depth of 70 feet (See Plates 31, 32, and 33). Eroded material was deposited in a bar approximately 600 feet downstream, the highest point being about 65 feet above the top of the sill.

Flow pattern and erosion studies were made for other combinations of crest gate openings (Plates 24 and 25). However, the condition with the right twelve gates wide open and the left two gates closed produced the most severe erosion. These tests indicated the desirability of operating the spillway gates in a uniform pattern in order to avoid unfavorable flow patterns and erosion conditions.

Test Results for Air Step

The purpose of the air step is to entrain or trap air on the underside of the nappe passing over the spillway crest. It is intended that the air remain within the sheet of water as it enters the spillway bucket and passes into the stilling basin. The air-water mixture thereby acts as a cushion for the flow in the bucket. In addition it is intended that the air act to reduce any tendency toward cavitation and pitting. Bureau of Reclamation studies indicate that in order to reduce cavitation pitting materially, it is

3. Ibid Page 12

necessary that the entrained air be in contact with the surface to be protected. As shown in Table IV, the maximum air entrainment in the Rihand model was 0.18% and occurred for an 8-foot gate opening with a discharge of 128,000 cfs. Under free fall conditions the maximum air entrainment was 0.13% and the minimum was 0.03% for discharges of 150,000 cfs and 418,000 cfs respectively. It should be noted that even these small percentages of entrainment were obtained only with a pressure under the nappe greater than atmospheric. When the pressure was maintained less than atmospheric, it was not possible to measure the quantity of air entrained. Under normal conditions the pressure must be less than atmospheric in order to have a flow into the underside of the nappe.

Although certain logical deductions can be made, it is not known exactly what laws of similitude are concerned in the phenomenon of air entrainment. Some air is carried along with the water by shearing action at the air and water interface. This drag may be primarily the result of viscous forces if the interface is extremely smooth, inertial forces if the interface is extremely rough, or a combination of both. If viscous forces are sufficiently great to enter the problem, then it is necessary to have the Reynolds number the same from the model to the prototype. This is impractical in ordinary model tests. If the roughness of the air-water interface is predominant, it is necessary only to have the roughness geometrically similar from the model to the prototype. This can be accomplished if the roughness is caused by the shape and size of the air step.

In the normal break-up of a jet there are two processes taking place. First, internal turbulence may come to the surface and cause boils and roughness at the air-water interface. Second, the drag of the air on the jet due to viscous forces causes wave action, similar to that on a lake, which eventually becomes unstable and breaks. The size and shape of waves and surface irregularities are determined to a large extent by surface tension as reflected in the Weber number.

From the foregoing discussion it is evident that the process of air entrainment is extremely complicated. In spite of the difficulty of making quantitative model studies of air entrainment, it is quite probable that the percent of air entrained in the prototype will not be more than 3 or 4 times that entrained in the model. This is still considerably less than the amount required, according to Bureau of Reclamation tests.

If it is possible to entrain the necessary amount of air at the prototype air step, there is still a question of its effect. That is, how much air will remain in the flow and how much will be in contact with the spillway face as the water enters the stilling basin. Air bubbles within the water are relatively smaller in the prototype than in the model. Therefore, the rise of air bubbles in the prototype is relatively slower. This would tend to increase the concentration of air near the bottom of the spillway in comparison with the concentration in the model.

The length of the air entrainment zone varied with the discharge and fluctuated with time, ranging from zero to approximately 80 ft. The length of the zone increased with discharge due to the increased length of the trajectory. Fluctuation with time was due to the sensitive balance of forces resulting perhaps from the use of an air step only 0.5 ft high and a crest shaped for atmospheric pressures under design head. If the height of the step was increased to 1.0 ft, it is possible that the zone would be more stable and that a slightly greater percentage of air would be entrained.

Chapter III

CONCLUSIONS AND RECOMMENDATIONS

Design

Overfall Spillway: The shape of the crest of the overfall spillway is satisfactory from the standpoint of protection against cavitation due to negative pressures and as an efficient design. Figures 12, 13, and 14 indicate that the positive pressures on the crest are not excessively high and that the sub-atmospheric pressures are of no significance.

However, at high discharges the spillway piers caused an excessive flow concentration and a depressed water surface, resulting in some reduction of discharge for the end bays. For maximum flows the nappe struck the training walls and spilled over. It is recommended that the design of the end pier nose be reviewed and modified to reduce the draw-down as much as possible. One method of solving the problem is to reduce the upstream projection of the end piers and to connect the upstream face of the dam to the pier face with as large a radius as possible.

The question of raising the spillway training walls cannot be answered definitely, because any change in end pier design will effect the flow on the spillway face. With no changes made in end piers, it may be that water spilling over the training walls at maximum flows is objectionable. If this is the case it is recommended that the training walls be raised 5 feet. This would give a freeboard of about 2 feet to take care of increased bulking of prototype flow.

Sluices: General performance of the sluice outlets was satisfactory. However, with no crest flow, improved performance in flow patterns under certain operating conditions would seem desirable. When either or both sluices were operating, a large whirl-pool action was set up in the stilling basin extending downstream beyond the end sill. Bed material was carried upstream onto the apron where it was kept in constant motion by the irregularities of flow. Resulting abrasive action will perhaps be negligible. The sluices in the prototype are expected to be operated only at rare intervals, and then for only short periods. Any modification of the spillway apron to improve flow conditions when operating the sluices, should be undertaken only in conjunction with model studies.

Stilling Basin: With the normal tailwater elevations anticipated in design, the stilling basin acted as an effective energy dissipator. However, when the tailwater depth was decreased 5 to 10 feet, the toe of the hydraulic jump moved a considerable distance downstream, and the jump came close to being swept off the apron. Even for normal tailwater, the jump formed further downstream than desirable for most economical use of the apron.

downstream, and the jump came close to being swept off the apron. Even for normal tailwater, the jump formed further downstream than desirable for most economical use of the apron.

It is recommended that consideration be given to lowering the apron. It is believed that if the apron were lowered about 10 feet, the length could be shortened 50 feet. The jump would become more stable with respect to changes in tailwater depth, and a quieter water surface would result in the downstream channel. It is also believed that the entire apron need not be lowered. The downstream end might be held at its present elevation, and the apron slope flattened to provide greater depth at the upstream end.

Consideration should also be given to lowering the stilling basin training walls. It is possible that the walls could be lowered 5 to 10 feet without detrimental effects on the action of the stilling basin.

The erosive tendencies downstream from the structure were minor. However, tests indicated that with subnormal tailwater depths, erosion increased due to less complete energy dissipation in the stilling basin. Unless there is a very stable channel control immediately downstream from the dam, there will be degradation of the channel by the water discharged from the reservoir where it has lost its sediment load.

Air Step: The air step in the model failed to entrain sufficient air to cause enthusiasm over its performance. Measurements indicated that only a fraction of one percent of the water discharge was air. Since the tests were conducted on a small scale model, they are not conclusive in showing how the prototype structure will perform. The model did show, however, that an air pocket formed beneath the nappe and below the air step. Undoubtedly some air entered the pocket beneath the nappe, but it was difficult to measure in the model. A greater percentage of air will unquestionably be entrained in the prototype than was entrained in the model. Knowledge about air entrainment is meager, and until more is known about the phenomenon, the test data may be studied for what it is worth. Engineering judgement and experience must at this time decide the practical value of the air step.

Prototype Operation

Certain rules should be followed in operating the prototype spillway structure. Model tests indicated that it would be possible to open the crest gates in increments up to four feet following any order of opening. However, for increments greater than four feet it would be preferable to follow some symmetrical pattern of opening. Such a pattern sequence could be: 2, 4, 6, 9, 12, 14, 13, 11, 10, 8, 7, 5, 3 and 1 or 6, 7, 8, 9, 12, 13, 14, 1, 2, 3, 4, 5, 11 and 10. As model tests have indicated, the hydraulic jump is fairly sensitive to changes in tailwater. In operating the gates, it would be desirable to limit the rate of increase in discharge such that the jump will always form on the apron. It is recommended that a thorough study of gate operation be made well in advance of a flood so that operators will be prepared to pass flood waters when necessary.

From past experience on other dams it has been found advisable to operate the spillway gates rather than the sluices, whenever the reservoir elevation permits. Discharges may be regulated by opening a single spillway gate any desired amount up to, say a four foot gate opening. Above this point the usual spillway operating procedure may be followed.

APPENDIX

LIST OF TABLES, FIGURES AND PLATES

TABLES

Table No.

FIGURES

Fig. No.

PLATES

Plate No.

Plate No.

22	Water Surface Profiles - Stilling Basin Tail Water 3.9 Feet Above Normal H W Elev 880.1, T W Elev 676.5, Q = 334,500 cfs
23	Flow Patterns - Stilling Basin Effect of Sluice Operation on Stilling Basin Performance H W Elev 880.5, T W Elev 664, Q = 69,500 cfs
24	Flow Patterns - Stilling Basin View from Downstream Gate Operation Schedule - Seven Right Gates Open 4 Feet H W Elev 880.1, T W Elev 639.6, Q = 51,400 cfs, Spillway and Powerhouse Flow Only
25	Flow Patterns - Stilling Basin Side View Gate Operation Schedule - Seven Right Gates Open 4 Feet H W Elev 880.1, T W Elev 639.6, Q = 51,400 cfs, Spillway and Powerhouse Flow Only
26	Flow Patterns - Stilling Basin Deposition of Bed Material Pattern Formed in the Downstream Channel Caused by Eddy H W Elev 880.5, T W Elev 644, Q = 69,500 cfs
27	Flow Patterns - Stilling Basin Gate Operation Schedule - Two Left Gates Closed H W Elev 888, T W Elev 685.5, Q = 390,000 cfs, Spillway Flow Only
28	Erosion Tests - River Channel Bed Movement Downstream from Stilling Basin - Side View H W Elev 887.6, T W Elev 679.7, Q = 477,000 cfs
29	Erosion Tests - River Channel Bed Movement Downstream from Stilling Basin Oblique View from Right Side H W Elev 887.6, T W Elev 679.7, Q = 477,000 cfs
30	Erosion Tests - River Channel Bed Movement Downstream from Stilling Basin Oblique View from Left Side H W Elev 887.6, T W Elev 679.7, Q = 477,000 cfs
31	Erosion Tests - River Channel Bed Movement Downstream from Stilling Basin View from Right Side Gate Operation Schedule - Two Left Gates Closed H W Elev 888, T W Elev 685.5, Q = 390,000 cfs, Spillway Flow Only

TABLE I

JUMP LOCATION ON APRON OF STILLING BASIN

H W El in ft	Tailwater El in ft	Tailwater Variation from Normal TW in ft	Discharge in cfs	Jump Position from Point of Tangency of Bucket in ft	Gate Opening
(1)	(2)	(3)	(4)	(5)	
887.6	689.7	+8.8	477,000	-55	Open*
887.6	679.7	-1.2	477,000	+15	Open*
887.6	674.7	-6.2	477,000	+60	Open*
887.6	669.7	-11.2	477,000	+75	Open*
860.6	638.2	-6.2	42,500	+25	Open**
879.9	637.9	-6.1	40,500	-15	2 ft**
880.2	640.8	-9.0	67,900	+40	4 ft**
880.3	644.6	-11.5	121,000	+87	8 ft**
878.6	633.1	-0.1	212,500	+5	16 ft**
880.1	676.5	+3.9	334,500	-60	Open*
880.1	666.5	-6.1	334,500	+15	Open*
880.1	661.5	-11.1	334,500	+70	Open*
880.1	656.5	-16.1	334,500	+105	Open*

Column (3)
(+) Designates greater than normal tailwater.
(-) Designates less than normal tailwater.

Column (5)
(+) Designates jump position is downstream from the point of tangency of the bucket.
(-) Designates jump position is upstream from the point of tangency of the bucket.

* Crest, sluices, and powerhouse operating.

** Crest flow only.

TABLE II

SURGE AND WAVE HEIGHTS

	Tailwater					
H W El in ft	El in ft	Variation from Normal T W in ft	Discharge in cfs	Average Surge in Tailrace in ft	Average Wave Height in River Channel in ft	Gate Opening
887.6	679.7	-1.2	477,000	2.74	4.32	Open
880.1	666.5	-6.1	334,500	2.02	4.03	Open
880.1	661.5	-11.1	334,500	1.87	3.31	Open
878.6	641.1	-8.7	68,200	0	1.22	2 ft
880.5	644.0	-6.0	69,500	0	1.08	2 ft
880.1	640.4	-1.2	28,500	0	0	Closed*

* Powerhouse and sluice discharge only

Location of measuring stations is shown on Fig. 11

TABLE III

DEPTH OF EROSION

H W El in ft	Tailwater El in ft	Tailwater Variation from Normal T W in ft	Discharge in cfs	Depth of Erosion Below Top of End Sill, in ft: End of Right Training Wall	Depth of Erosion Below Top of End Sill, in ft: End of Left Training Wall	Maximum Height of Deposition Above top of End Sill, in ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)
888.0	685.5	+9.3	390,000*	70	-	65
887.6	679.7	-1.2	477,000	32	17	47
880.1	666.5	-6.1	334,500	11	13	36
880.1	661.5	-11.1	334,500	11	11	31
878.6	641.1	-8.7	68,200**	10	8	27
880.1	640.4	-1.2	28,500***	-7	6	23

Column (4)
* Flow through the right 12 spillway bays only
** 2-ft gate opening
*** Powerhouse and sluice discharge only

Column (5)
(-) Designates deposition at this point rather than degradation

TABLE IV

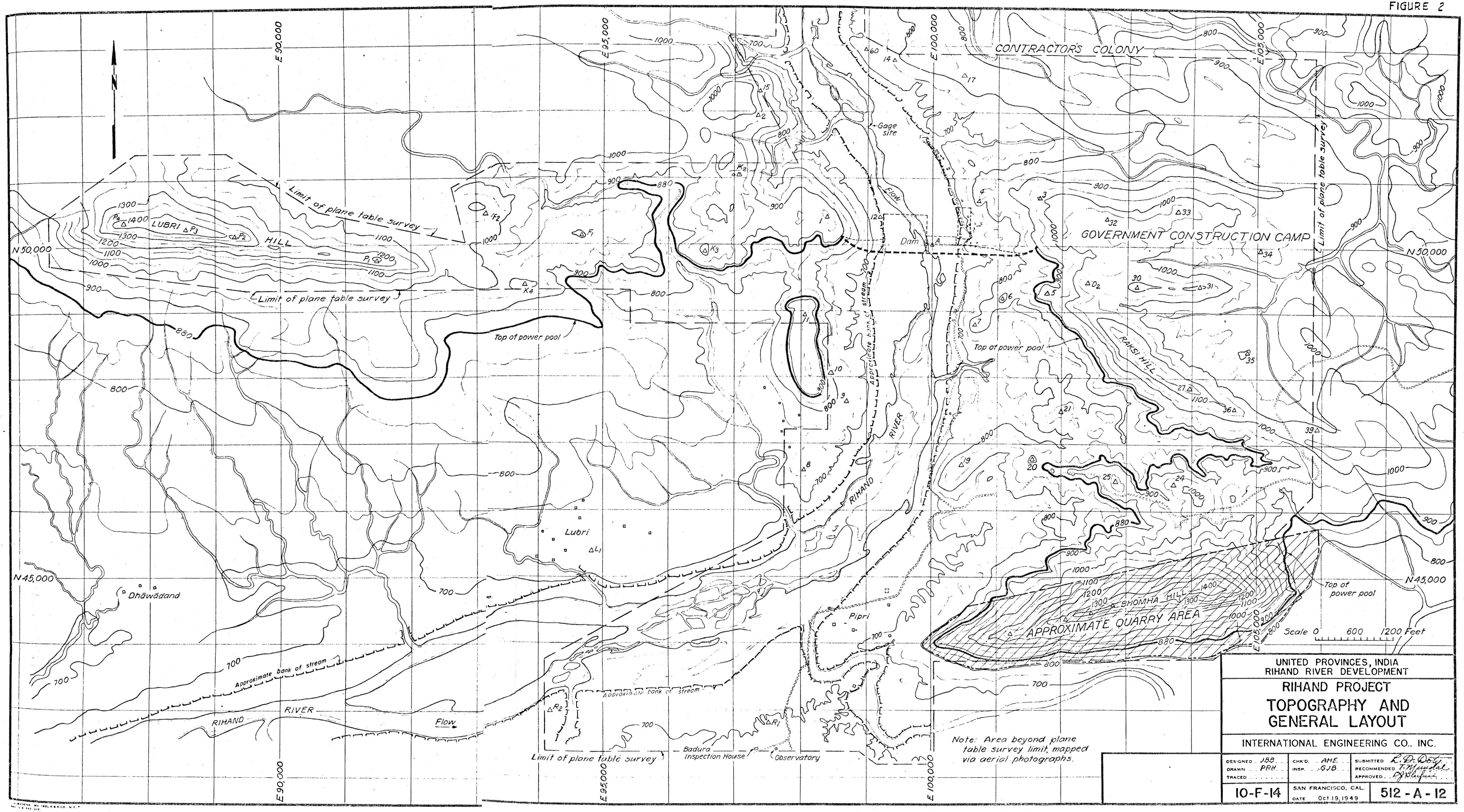
AIR STEP TESTS

H W El ft	Qw cfs	Pressure Under Nappe*	(Qa)m cfs	Qa cfs	Qa/Qw Percent
878.8 (Gates open)	282,000	0.30	0.00692	305.0	0.110
		0.60	0.00566	249.0	0.090
		0.40	0.00439	193.0	0.070
		0.40	0.00358	157.5	0.060
		0.30	0.00253	111.0	0.040
886.2 (Gates open)	418,000	0.50	0.00566	249.0	0.060
		0.50	0.00506	223.0	0.050
		0.40	0.00439	193.0	0.050
		0.50	0.00358	157.5	0.050
		0.20	0.00253	111.0	0.030
870.3 (Gates open)	150,000	0.20	0.00439	193.0	0.130
		0.40	0.00400	176.0	0.120
		0.30	0.00358	157.5	0.105
		0.30	0.00358	157.5	0.105
883.5 (8-ft gate opening)	128,000	1.20	0.00536	236.0	0.180
		0.60	0.00474	208.0	0.160
		0.70	0.00420	185.0	0.140
		0.40	0.00357	157.5	0.120

* Pressure under the nappe, downstream from the air step, measured in feet of water prototype. All readings are above atmospheric pressure.

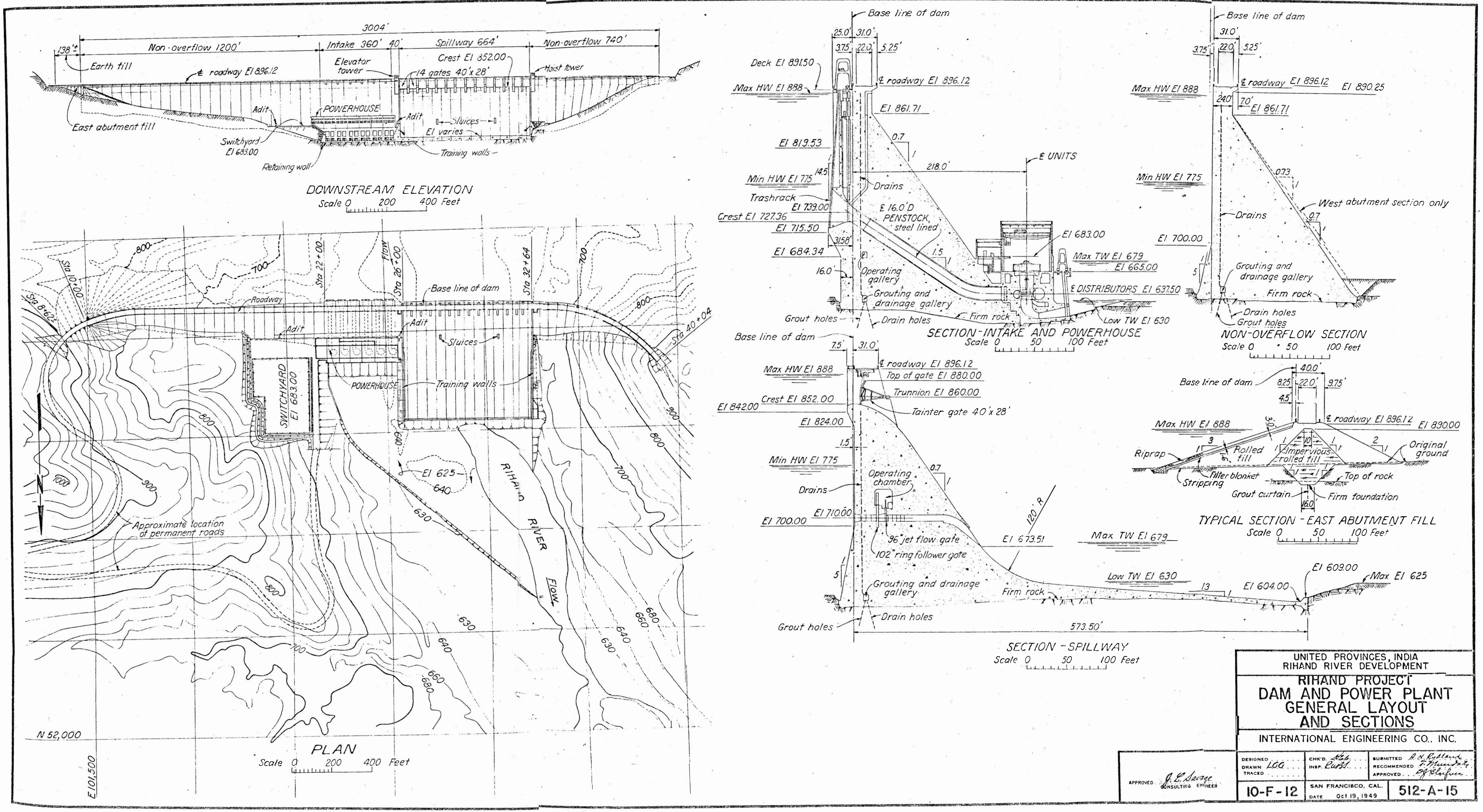
(Qa)m = Air discharge in model.

FIGURE 2



CONTRACTOR'S COLONY
GOVERNMENT CONSTRUCTION CAMP
LUBRI HILL
RAKSI HILL
BHOMHA HILL
APPROXIMATE QUARRY AREA
Limit of plane table survey
Top of power pool
Approximate bank of stream
RIHAND RIVER
Flow
Gage site
Dam
Lubri
Dhāwādand
Pipri
Badura Inspection House
Observatory
N 50,000
N 45,000
E 90,000
E 95,000
E 100,000
E 105,000
Scale 0 600 1200 Feet
Note: Area beyond plane table survey limit mapped via aerial photographs.
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
TOPOGRAPHY AND GENERAL LAYOUT
INTERNATIONAL ENGINEERING CO., INC.
DESIGNED JBB
DRAWN PRH
TRACED
CHK'D. AHE
INSP. GJB
SUBMITTED
RECOMMENDED
APPROVED
10-F-14
SAN FRANCISCO, CAL.
DATE Oct.19,1949
512-A-12

FIGURE 3


3004'
138'±
Non-overflow 1200'
Intake 360'
40'
Spillway 664'
Non-overflow 740'
Earth fill
℄ roadway El 896.12
Elevator tower
Crest El 852.00
14 gates 40' x 28'
Hoist tower
Adit
POWERHOUSE
Sluices
El varies
East abutment fill
Switchyard El 683.00
Retaining wall
Training walls
DOWNSTREAM ELEVATION
Scale 0 200 400 Feet
Sta 10+00
Sta 8+62
Sta 22+00
Sta 26+00
Sta 32+64
Sta 40+04
Flow
Base line of dam
Roadway
SWITCHYARD El 683.00
El 625
RIHAND RIVER
Approximate location of permanent roads
N 52,000
E 101,500
PLAN
Scale 0 200 400 Feet
Base line of dam
25.0'
31.0'
3.75
22.0'
5.25'
Deck El 891.50
Max HW El 888
℄ roadway El 896.12
El 861.71
El 819.53
℄ UNITS
218.0'
Min HW El 775
Drains
Trashrack
El 739.00
Crest El 727.36
El 715.50
℄ 16.0'D PENSTOCK, steel lined
El 683.00
El 684.34
Max TW El 679
El 665.00
16.0'
Operating gallery
Grouting and drainage gallery
℄ DISTRIBUTORS El 637.50
Grout holes
Drain holes
Firm rock
Low TW El 630
SECTION-INTAKE AND POWERHOUSE
Scale 0 50 100 Feet
Base line of dam
31.0'
3.75'
22.0'
5.25'
Max HW El 888
℄ roadway El 896.12
El 890.25
El 861.71
Min HW El 775
West abutment section only
Drains
El 700.00
Grouting and drainage gallery
Firm rock
Drain holes
Grout holes
NON-OVERFLOW SECTION
Scale 0 50 100 Feet
Base line of dam
7.5'
31.0'
Max HW El 888
℄ roadway El 896.12
Top of gate El 880.00
Trunnion El 860.00
Crest El 852.00
El 842.00
Tainter gate 40' x 28'
El 824.00
Min HW El 775
Operating chamber
Drains
El 710.00
El 700.00
96" jet flow gate
102" ring follower gate
120' R
El 673.51
Max TW El 679
Low TW El 630
El 609.00
Max El 625
El 604.00
Grouting and drainage gallery
Firm rock
Grout holes
Drain holes
573.50'
SECTION - SPILLWAY
Scale 0 50 100 Feet
40.0'
Base line of dam
8.25
22.0'
9.75'
℄ roadway El 896.12
El 890.00
Max HW El 888
Riprap
Rolled fill
Impervious rolled fill
Original ground
Filter blanket
Stripping
Top of rock
Grout curtain
Firm foundation
16.0
TYPICAL SECTION - EAST ABUTMENT FILL
Scale 0 50 100 Feet
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
DAM AND POWER PLANT
GENERAL LAYOUT
AND SECTIONS
INTERNATIONAL ENGINEERING CO., INC.
APPROVED J. L. Savage CONSULTING ENGINEER
DESIGNED
DRAWN LGG
TRACED
SUBMITTED A. N. Rydland
10-F-12
SAN FRANCISCO, CAL.
DATE Oct 19, 1949
512-A-15

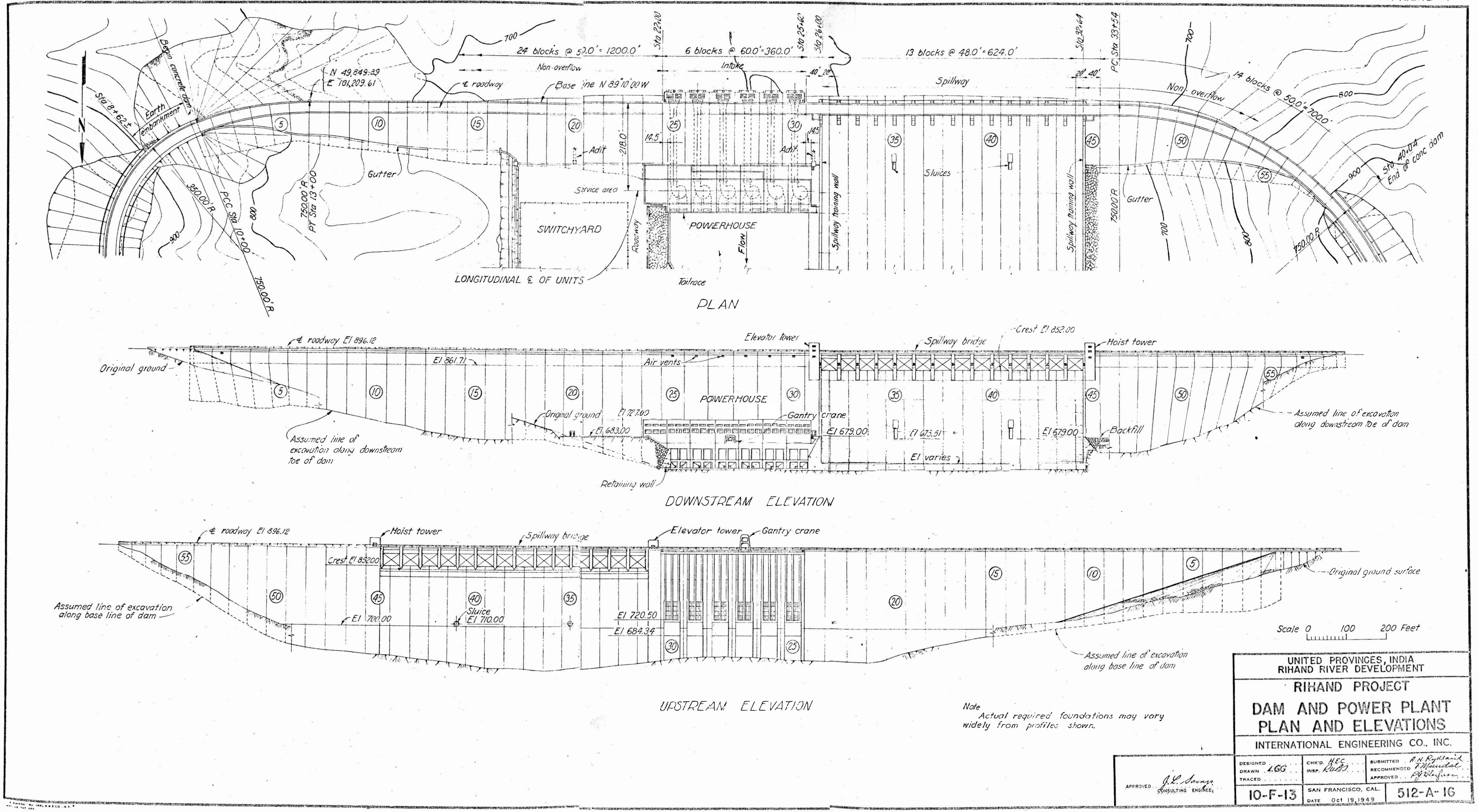
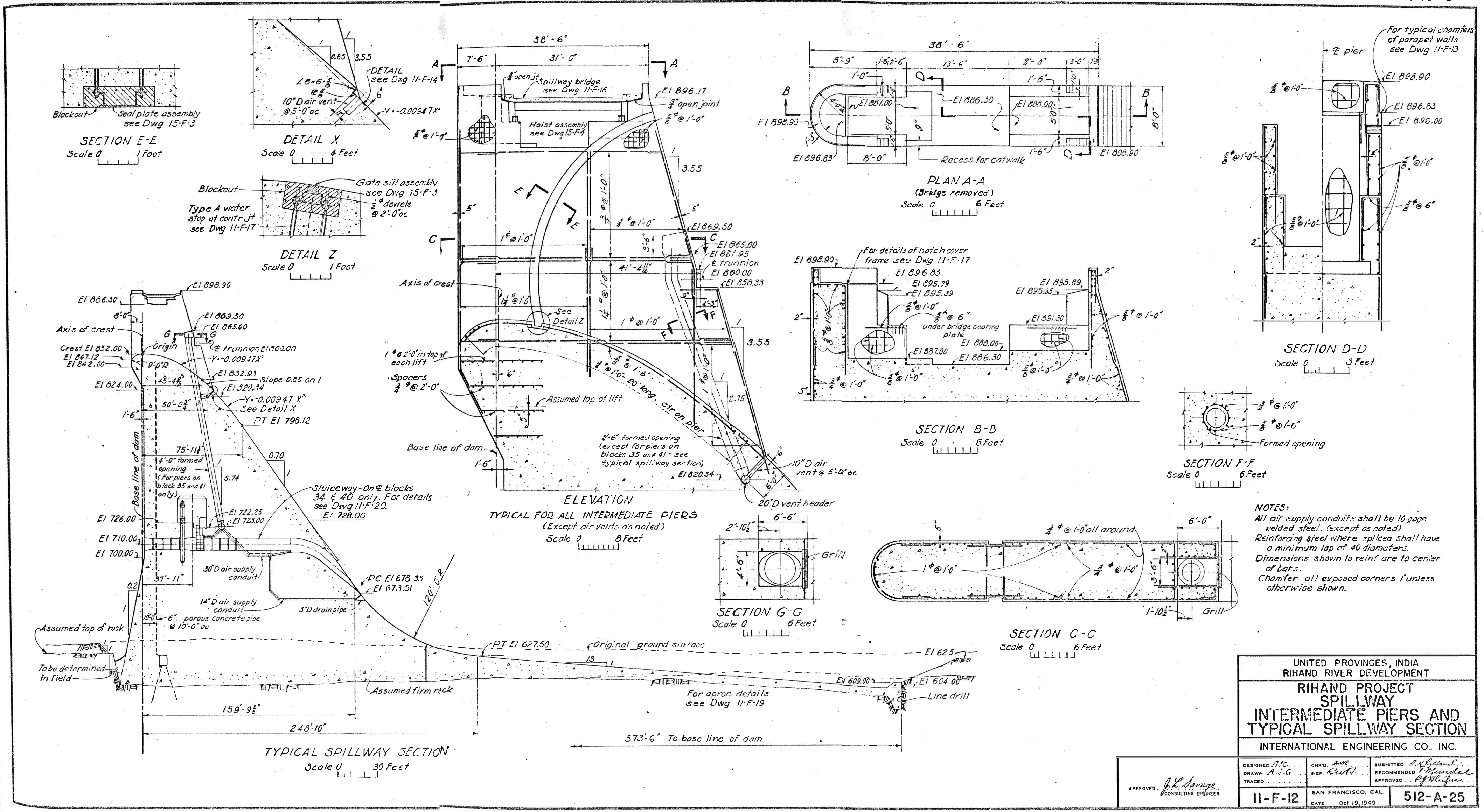
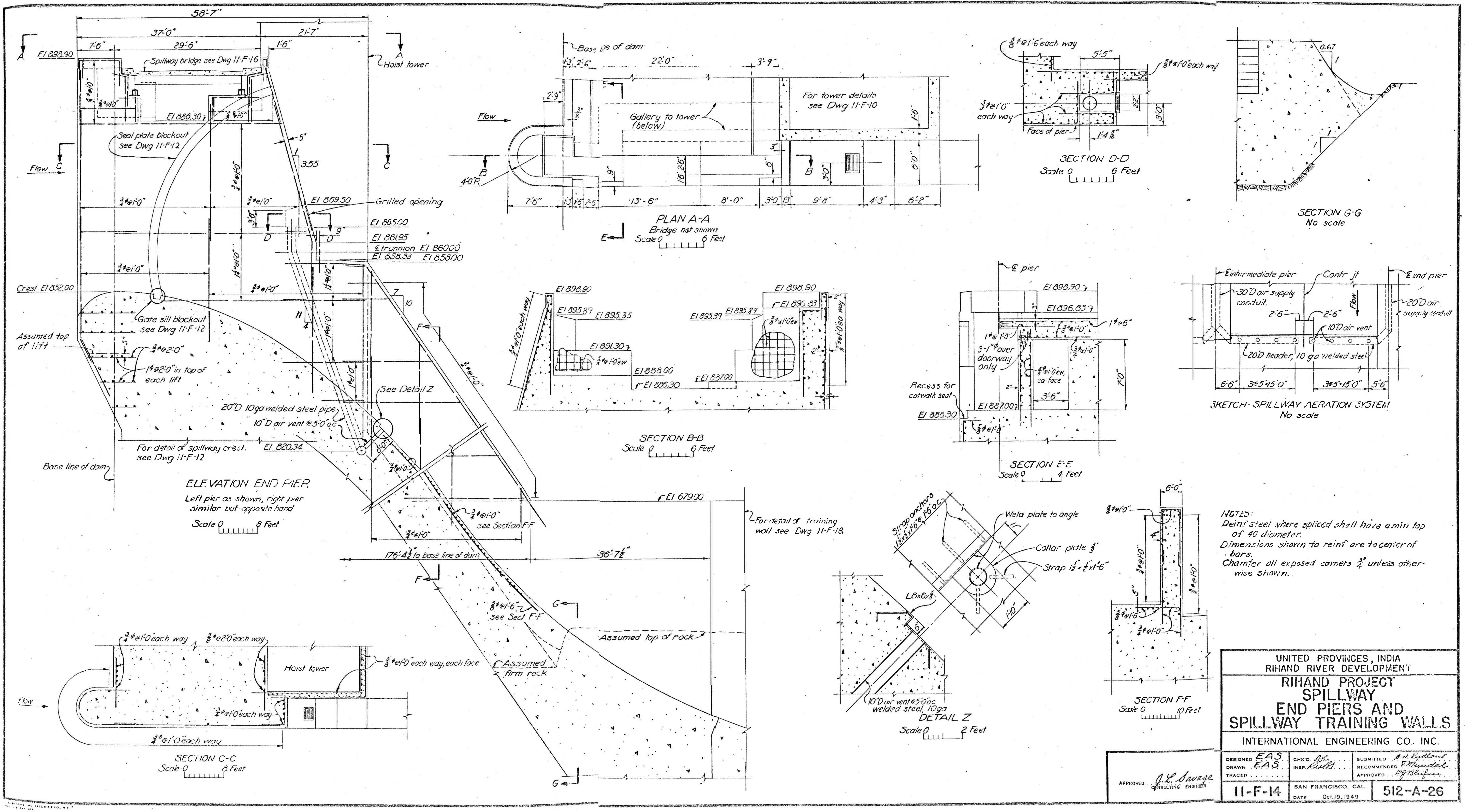

24 blocks @ 50.0' = 1200.0'
6 blocks @ 60.0' = 360.0'
13 blocks @ 48.0' = 624.0'
14 blocks @ 50.0' = 700.0'
Sta 22+00
Sta 25+62
Sta 26+00
Sta 32+64
PC Sta 33+54
Non-overflow
Intake
Spillway
Non overflow
Begin concrete dam
Earth embankment
Sta 8+62±
N 49,849.39
E 101,209.61
℄ roadway
Base line N 89°10'00"W
Adit
Gutter
Service area
SWITCHYARD
Roadway
POWERHOUSE
Flow
Tailrace
Spillway training wall
Sluices
Sta 40+04 End of conc dam
750.00'R
PT Sta 13+00
350.00'R
PCC Sta 10+00
LONGITUDINAL ℄ OF UNITS
PLAN
℄ roadway El 896.12
Original ground
El 861.71
Air vents
Elevator tower
Spillway bridge
Crest El 852.00
Hoist tower
POWERHOUSE
Gantry crane
El 727.00
El 683.00
El 679.00
El 673.51
El varies
Backfill
Assumed line of excavation along downstream toe of dam
Retaining wall
DOWNSTREAM ELEVATION
Hoist tower
Spillway bridge
Elevator tower
Gantry crane
Crest El 852.00
Sluice El 710.00
El 700.00
El 720.50
El 684.34
Original ground surface
Assumed line of excavation along base line of dam
UPSTREAM ELEVATION
Scale 0 100 200 Feet
Note
Actual required foundations may vary widely from profiles shown.
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
DAM AND POWER PLANT
PLAN AND ELEVATIONS
INTERNATIONAL ENGINEERING CO., INC.
DESIGNED
DRAWN LGG
TRACED
CHK'D. HEC
INSP. RAB
SUBMITTED A.N. Rydland
RECOMMENDED
APPROVED
APPROVED J.L. Savage CONSULTING ENGINEER
10-F-13
SAN FRANCISCO, CAL.
DATE Oct 19, 1949
512-A-16

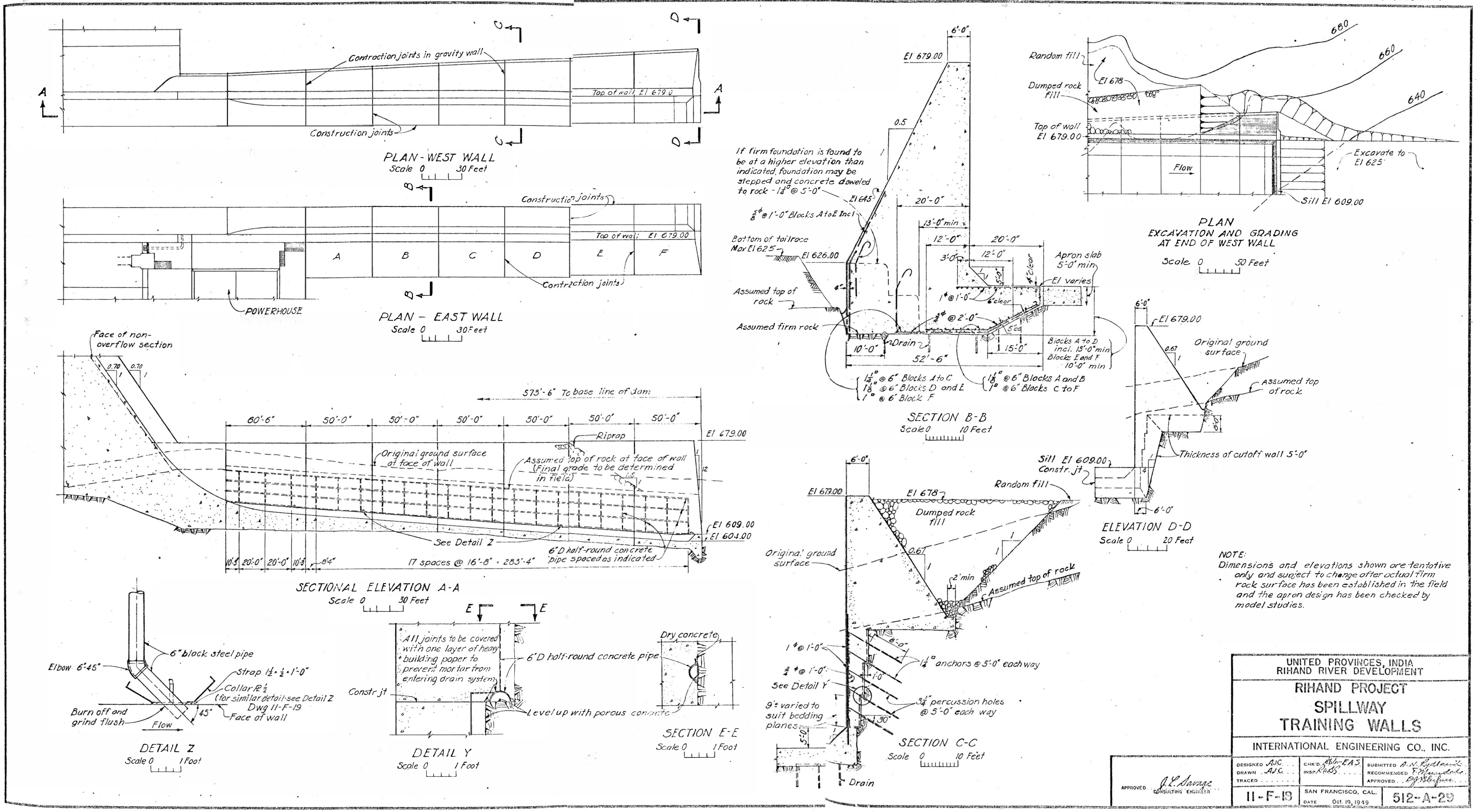
FIGURE 5


SECTION E-E
Scale 0 1 Foot
Blockout
Seal plate assembly see Dwg 15-F-3
DETAIL X
Scale 0 4 Feet
DETAIL see Dwg 11-F-14
10" D air vent @ 5'-0" oc
Y = -0.00947X²
DETAIL Z
Scale 0 1 Foot
Blockout
Gate sill assembly see Dwg 15-F-3
1/2 φ dowels @ 2'-0" oc
Type A water stop at contr jt see Dwg 11-F-17
ELEVATION
TYPICAL FOR ALL INTERMEDIATE PIERS
(Except air vents as noted)
Scale 0 8 Feet
Spillway bridge see Dwg 11-F-16
Hoist assembly see Dwg 15-F4
Axis of crest
Base line of dam
Assumed top of lift
2'-6" formed opening (except for piers on blocks 35 and 41 - see typical spillway section)
10" D air vent @ 5'-0" oc
20" D vent header
PLAN A-A
(Bridge removed)
Scale 0 6 Feet
Recess for catwalk
SECTION B-B
Scale 0 6 Feet
For details of hatch cover frame see Dwg 11-F-17
SECTION D-D
Scale 0 3 Feet
For typical chamfers of parapet walls see Dwg 11-F-13
SECTION F-F
Scale 0 6 Feet
Formed opening
SECTION G-G
Scale 0 6 Feet
Grill
SECTION C-C
Scale 0 6 Feet
TYPICAL SPILLWAY SECTION
Scale 0 30 Feet
Sluiceway - On ℄ blocks 34 & 40 only. For details see Dwg 11-F-20.
30" D air supply conduit
14" D air supply conduit
6" porous concrete pipe @ 10'-0" oc
3" D drain pipe
Assumed top of rock
To be determined in field
Assumed firm rock
Original ground surface
For apron details see Dwg 11-F-19
Line drill
573'-6" To base line of dam
NOTES:
All air supply conduits shall be 10 gage welded steel. (except as noted)
Reinforcing steel where spliced shall have a minimum lap of 40 diameters.
Dimensions shown to reinf are to center of bars.
Chamfer all exposed corners 1" unless otherwise shown.
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
SPILLWAY
INTERMEDIATE PIERS AND
TYPICAL SPILLWAY SECTION
INTERNATIONAL ENGINEERING CO., INC.
APPROVED J. L. Savage CONSULTING ENGINEER
11-F-12
SAN FRANCISCO, CAL.
DATE Oct. 19, 1949
512-A-25



58'-7"
37'-0"
21'-7"
7'-6"
29'-6"
1'-6"
El 898.90
Spillway bridge see Dwg 11-F-16
A Hoist tower
El 886.30
Seal plate blockout see Dwg 11-F-12
Flow C
C
3.55
El 869.50
Grilled opening
El 865.00
El 861.95
℄ trunnion El 860.00
El 858.33
El 858.00
Crest El 852.00
Gate sill blockout see Dwg 11-F-12
Assumed top of lift
1ϕ@2'-0" in top of each lift
See Detail Z
20"D 10ga welded steel pipe
10"D air vent @ 5'-0" o.c.
El 820.34
For detail of spillway crest, see Dwg 11-F-12
Base line of dam
ELEVATION END PIER
Left pier as shown, right pier similar but opposite hand
Scale 0 8 Feet
El 679.00
For detail of training wall see Dwg 11-F-18
see Section F-F
176'-4½" to base line of dam
36'-7½"
see Sect F-F
Assumed top of rock
Assumed firm rock
Base line of dam
22'-0"
3'-9"
Flow
For tower details see Dwg 11-F-10
Gallery to tower (below)
4'-0"R
7'-6"
13'-6"
8'-0"
9'-8"
4'-3"
6'-2"
PLAN A-A
Bridge not shown
Scale 0 6 Feet
SECTION D-D
Scale 0 6 Feet
Face of pier
SECTION G-G
No scale
0.67
El 898.90
El 895.89
El 895.35
El 891.30
El 888.00
El 886.30
El 887.00
El 895.39
El 896.83
SECTION B-B
Scale 0 6 Feet
℄ pier
El 896.83
3-1"ϕ over doorway only
Recess for catwalk seat
El 888.30
El 887.00
SECTION E-E
Scale 0 4 Feet
℄ intermediate pier
Contr jt
℄ end pier
30"D air supply conduit.
20"D air supply conduit
10"D air vent
20"D header, 10 ga welded steel
6'-6"
3@5'-15'-0"
3@5'-15'-0"
5'-6"
SKETCH-SPILLWAY AERATION SYSTEM
No scale
Strap anchors
Weld plate to angle
Collar plate ⅝"
L8x6x⅝
10"D air vent @ 5'-0" oc welded steel, 10ga
DETAIL Z
Scale 0 2 Feet
6'-0"
SECTION F-F
Scale 0 10 Feet
NOTES:
Reinf steel where spliced shall have a min lap of 40 diameter.
Dimensions shown to reinf are to center of bars.
Chamfer all exposed corners ¾" unless otherwise shown.
Hoist tower
Flow
SECTION C-C
Scale 0 8 Feet
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
SPILLWAY
END PIERS AND
SPILLWAY TRAINING WALLS
INTERNATIONAL ENGINEERING CO., INC.
DESIGNED EAS
DRAWN EAS
TRACED
APPROVED J. L. Savage CONSULTING ENGINEER
SUBMITTED
RECOMMENDED
APPROVED
11-F-14
SAN FRANCISCO, CAL.
DATE Oct.19,1949
512-A-26

FIGURE 7



Contraction joints in gravity wall
Top of wall El 679.0
Construction joints
PLAN - WEST WALL
Scale 0 30 Feet
Construction joints
Top of wall El 679.00
A B C D E F
Contraction joints
POWERHOUSE
PLAN - EAST WALL
Scale 0 30 Feet
Face of non-overflow section
573'-6" To base line of dam
60'-6" 50'-0" 50'-0" 50'-0" 50'-0" 50'-0" 50'-0"
El 679.00
Riprap
Original ground surface at face of wall
Assumed top of rock at face of wall (Final grade to be determined in field)
See Detail Z
6"D half-round concrete pipe spaced as indicated
El 609.00
El 604.00
10'-5" 20'-0" 20'-0" 10'-5" 8'-4" 17 spaces @ 16'-8" = 283'-4"
SECTIONAL ELEVATION A-A
Scale 0 30 Feet
6" black steel pipe
Elbow 6"-45°
Strap 1½ x ½ x 1'-0"
Collar ℞ ½ (for similar detail-see Detail Z Dwg 11-F-19
Burn off and grind flush
Flow
45°
Face of wall
DETAIL Z
Scale 0 1 Foot
All joints to be covered with one layer of heavy building paper to prevent mortar from entering drain system
6"D half-round concrete pipe
Constr jt
Level up with porous concrete
DETAIL Y
Scale 0 1 Foot
Dry concrete
SECTION E-E
Scale 0 1 Foot
El 679.00
6'-0"
0.5
If firm foundation is found to be at a higher elevation than indicated, foundation may be stepped and concrete doweled to rock - 1¼" @ 5'-0"
El 645
⅝"φ @ 1'-0" Blocks A to E Incl
20'-0"
13'-0" min
12'-0" 20'-0"
Bottom of tailrace Max El 625
El 626.00
3'-0" 12'-0"
Apron slab 5'-0" min
El varies
Assumed top of rock
1"φ @ 1'-0"
¾"φ @ 2'-0"
Assumed firm rock
Drain
10'-0" 52'-6" 15'-0"
Blocks A to D incl. 13'-0" min Blocks E and F 10'-0" min
1¼" @ 6" Blocks A to C
1⅛" @ 6" Blocks D and E
1" @ 6" Block F
1⅛" @ 6" Blocks A and B
1" @ 6" Blocks C to F
SECTION B-B
Scale 0 10 Feet
Random fill
El 678
Dumped rock fill
Top of wall El 679.00
680 660 640
Flow
Excavate to El 625
Sill El 609.00
PLAN
EXCAVATION AND GRADING AT END OF WEST WALL
Scale 0 50 Feet
6'-0"
El 679.00
0.67
Original ground surface
Assumed top of rock
Thickness of cutoff wall 5'-0"
Sill El 609.00
Constr. jt
6'-0"
ELEVATION D-D
Scale 0 20 Feet
6'-0"
El 679.00
El 678
Random fill
Dumped rock fill
Original ground surface
0.67
2' min
Assumed top of rock
1"φ @ 1'-0"
¾"φ @ 1'-0"
See Detail Y
9'± varied to suit bedding planes
1¼" anchors @ 5'-0" each way
3¼" percussion holes @ 5'-0" each way
30°
Drain
SECTION C-C
Scale 0 10 Feet
NOTE:
Dimensions and elevations shown are tentative only and subject to change after actual firm rock surface has been established in the field and the apron design has been checked by model studies.
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
SPILLWAY
TRAINING WALLS
INTERNATIONAL ENGINEERING CO., INC.
DESIGNED AJC
DRAWN AJC
TRACED
CHK'D EAS
INSP
SUBMITTED
RECOMMENDED
APPROVED
APPROVED J. L. Savage CONSULTING ENGINEER
11-F-18
SAN FRANCISCO, CAL.
DATE Oct. 19, 1949
512-A-29

FIGURE 8

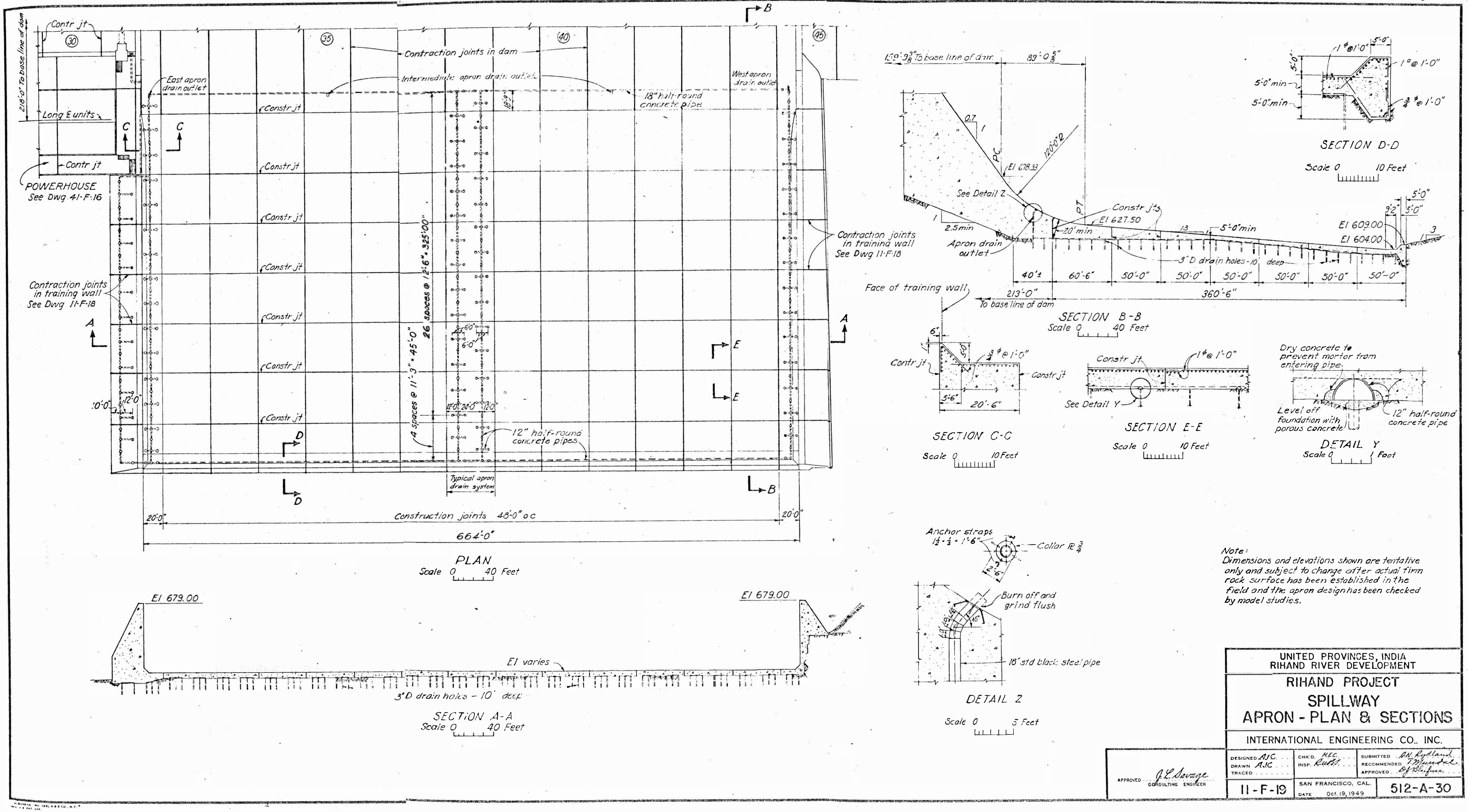
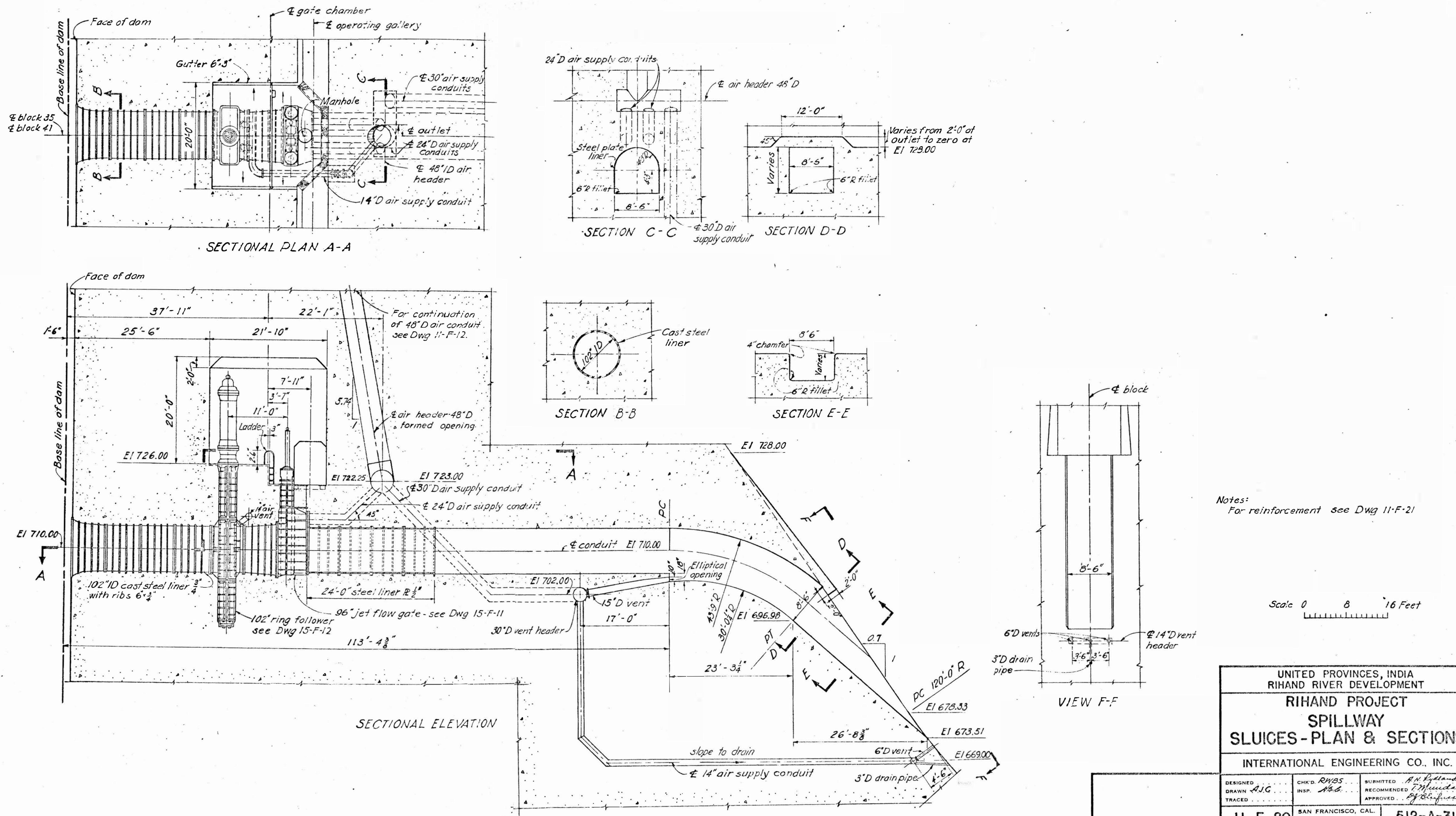

Contr jt
Contraction joints in dam
Intermediate apron drain outlet
East apron drain outlet
West apron drain outlet
18" half-round concrete pipe
218'-0" To base line of dam
Long ℄ units
Constr jt
POWERHOUSE
See Dwg 41-F-16
Contraction joints in training wall
See Dwg 11-F-18
4 spaces @ 11'-3" = 45'-0"
26 spaces @ 12'-6" = 325'-00"
12" half-round concrete pipes
Typical apron drain system
Construction joints 48'-0" o c
664'-0"
PLAN
Scale 0 40 Feet
El 679.00
El varies
3" D drain holes - 10' deep
SECTION A-A
Scale 0 40 Feet
159'-9 3/8" To base line of dam
89'-0 5/8"
120'-0" R
El 678.33
See Detail Z
Apron drain outlet
Constr jts
El 627.50
20' min
5'-0" min
3" D drain holes - 10' deep
El 609.00
El 604.00
40'±
60'-6"
50'-0"
213'-0"
To base line of dam
360'-6"
SECTION B-B
Scale 0 40 Feet
Face of training wall
Contr jt
Constr jt
5'-6"
20'-6"
SECTION C-C
Scale 0 10 Feet
SECTION D-D
Scale 0 10 Feet
1" ϕ @ 1'-0"
3/4" ϕ @ 1'-0"
5'-0" min
See Detail Y
SECTION E-E
Scale 0 10 Feet
Dry concrete to prevent mortar from entering pipe
Level off foundation with porous concrete
12" half-round concrete pipe
DETAIL Y
Scale 0 1 Foot
Anchor straps 1½ × ½ × 1'-6"
Collar ℞ 3/4
Burn off and grind flush
18" std black steel pipe
DETAIL Z
Scale 0 5 Feet
Note:
Dimensions and elevations shown are tentative only and subject to change after actual firm rock surface has been established in the field and the apron design has been checked by model studies.
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT
RIHAND PROJECT
SPILLWAY
APRON - PLAN & SECTIONS
INTERNATIONAL ENGINEERING CO., INC.
DESIGNED AJC
DRAWN AJC
TRACED
CHK'D H.E.C.
INSP.
SUBMITTED
RECOMMENDED
APPROVED
APPROVED J.L. Savage CONSULTING ENGINEER
11-F-19
SAN FRANCISCO, CAL.
DATE Oct. 19, 1949
512-A-30

FIGURE 9



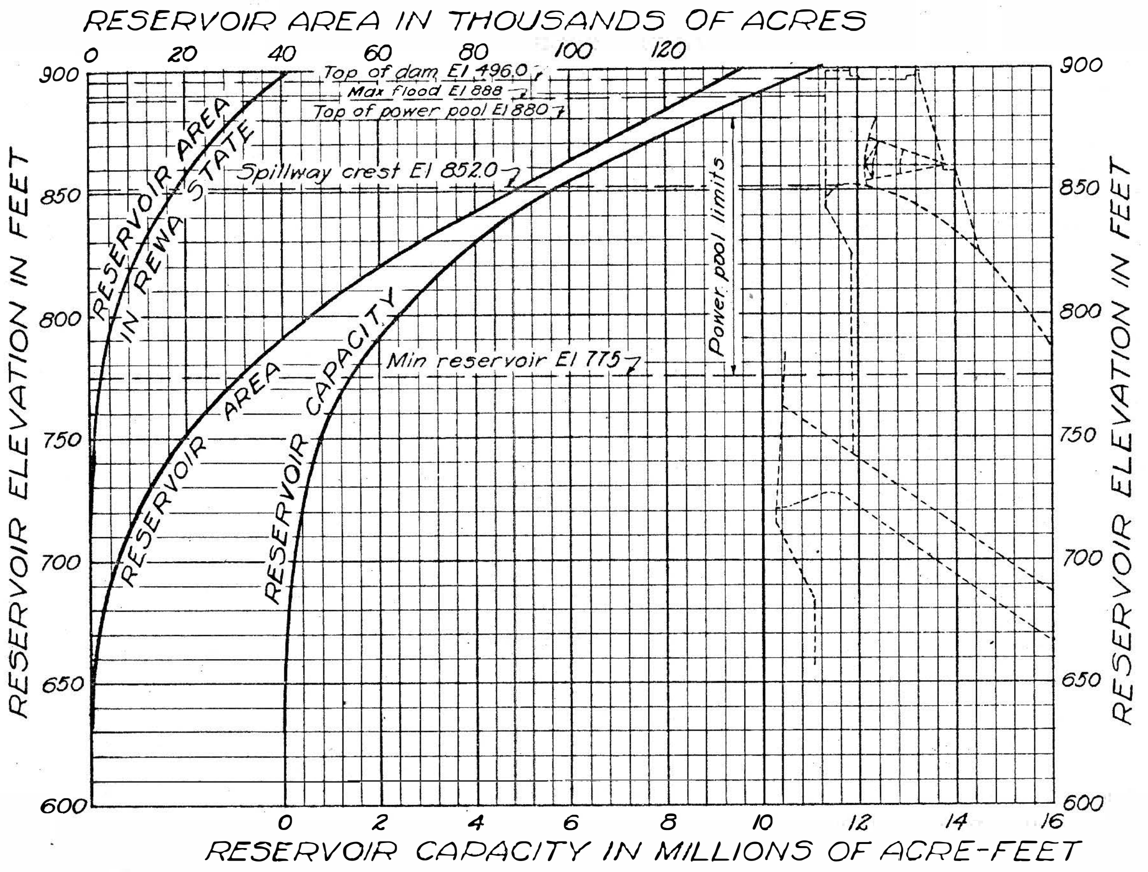
UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT

RIHAND PROJECT
SPILLWAY
SLUICES-PLAN & SECTIONS

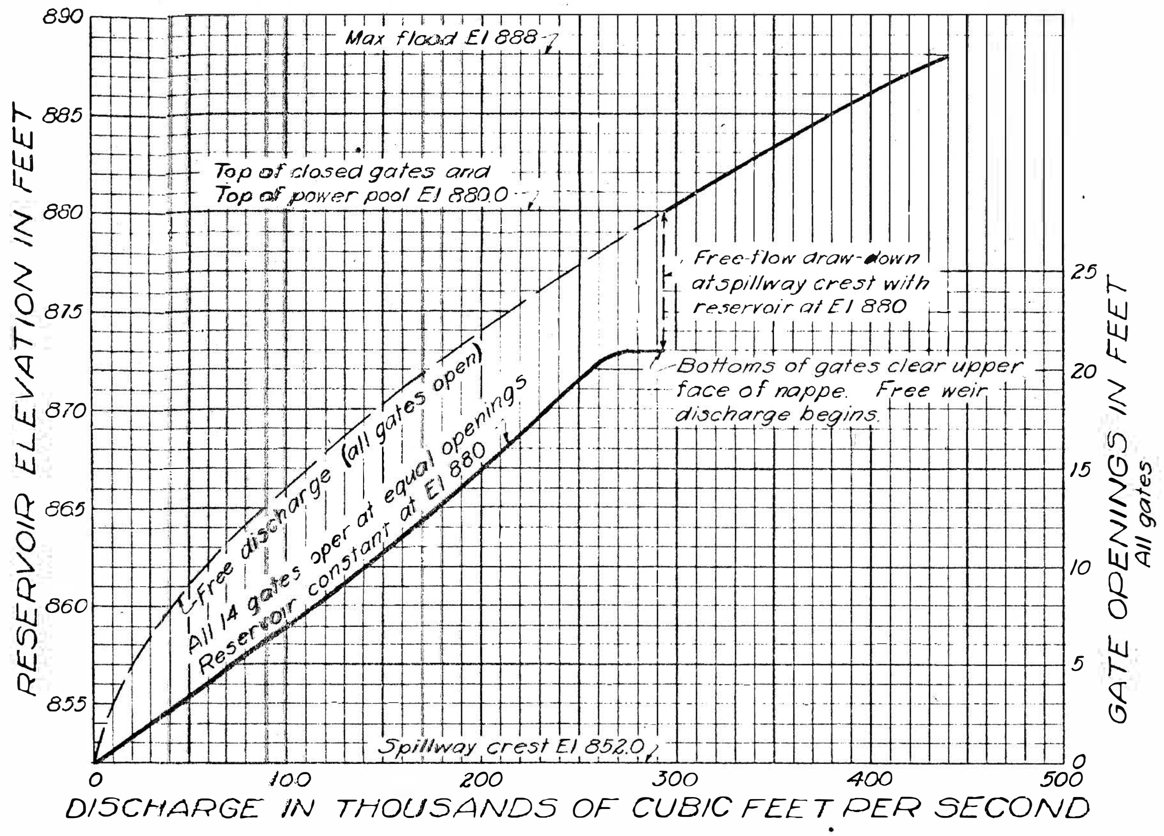
INTERNATIONAL ENGINEERING CO., INC.

DESIGNED ...... CHK'D. RWBS SUBMITTED A.H. Rydland
DRAWN AJC INSP. ... RECOMMENDED ...
TRACED ...... APPROVED ...

11-F-20 SAN FRANCISCO, CAL. DATE Oct 19, 1949 512-A-31

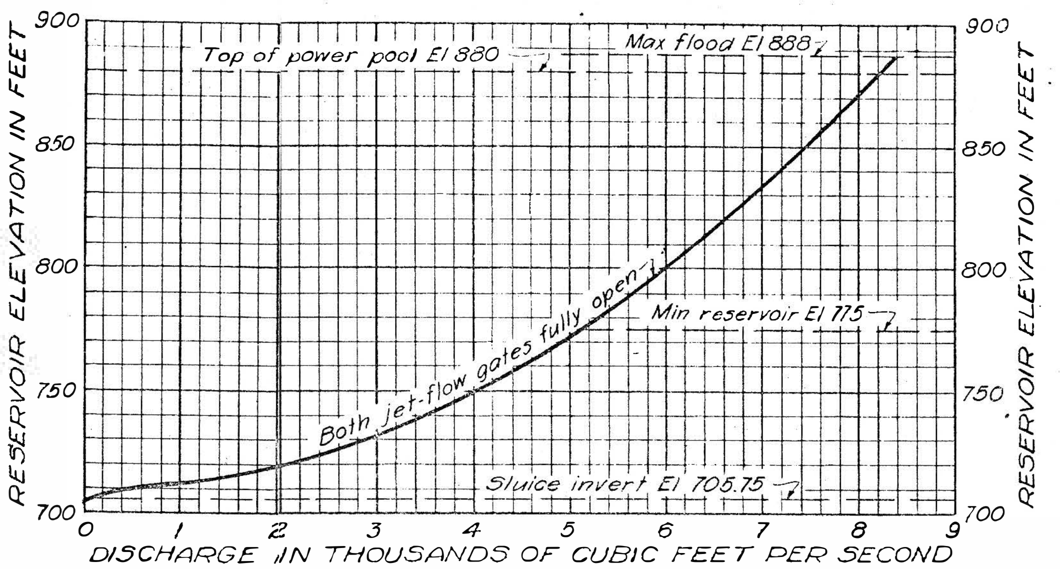


AREA AND CAPACITY CURVES



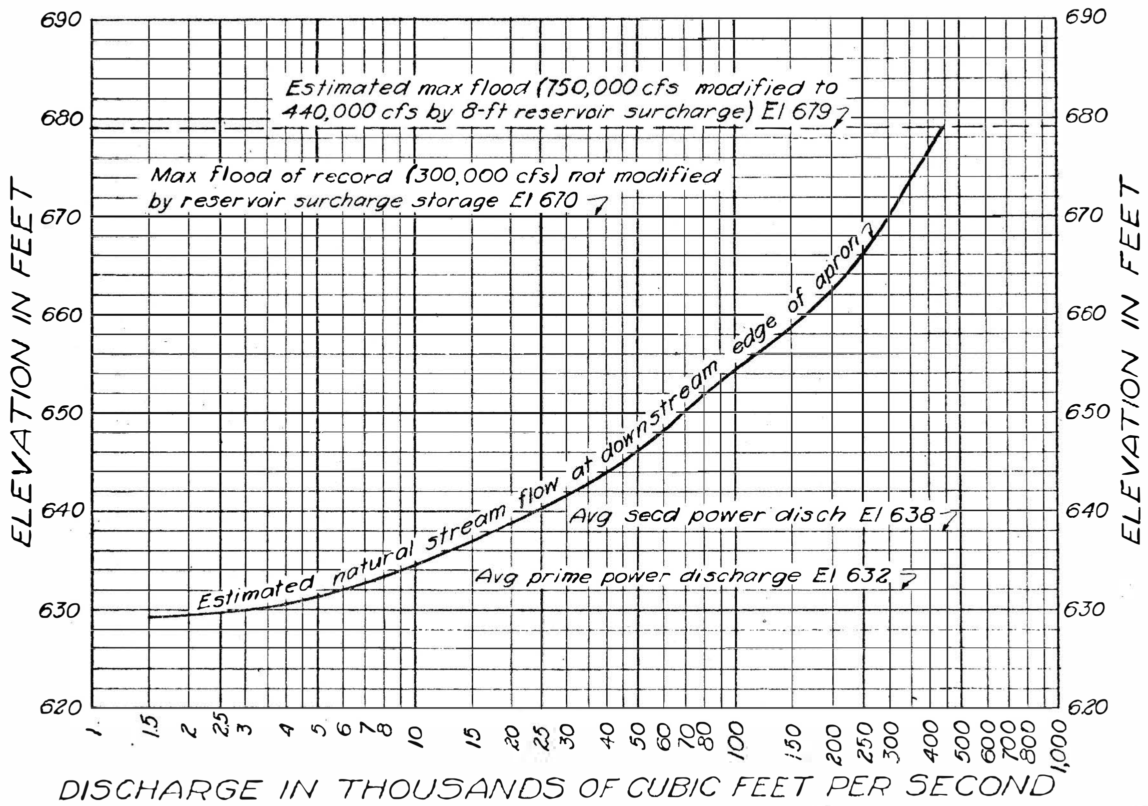
SPILLWAY RATING CURVE

Fourteen 40' x 28' Tainter gates operating simultaneously

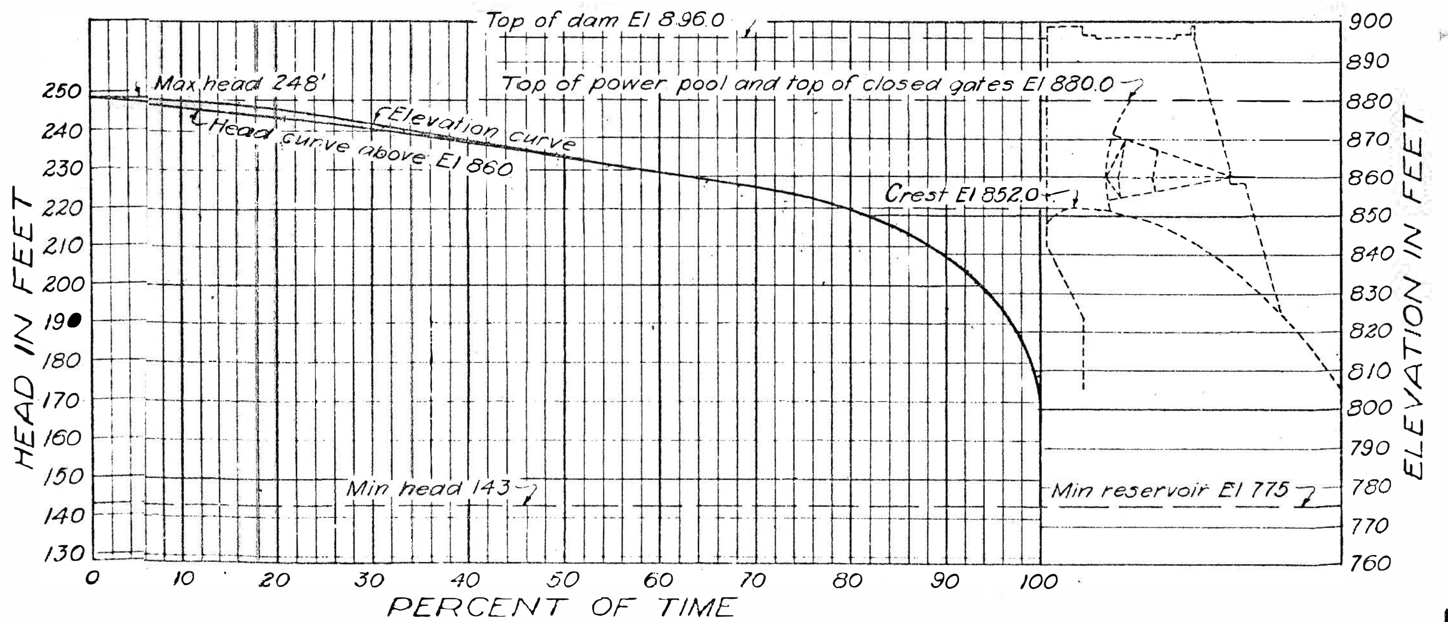


SLUICE RATING CURVE

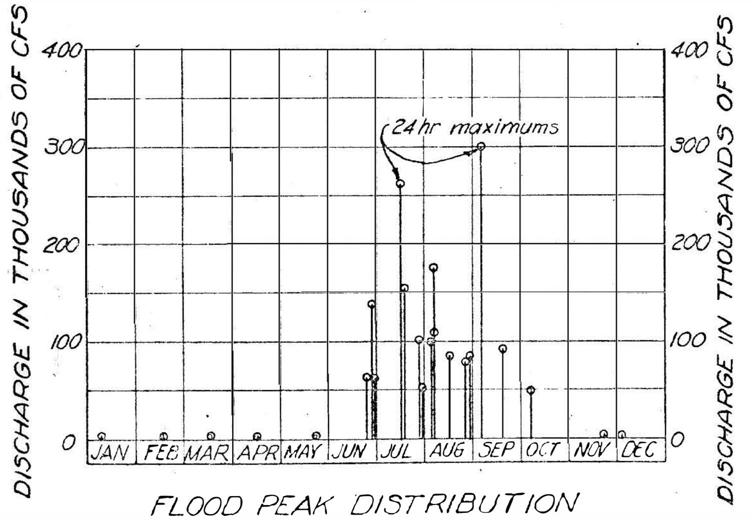
Two jet-flow sluice gates



TAILWATER RATING CURVE



HEAD AND ELEVATION DURATION CURVES



FLOOD PEAK DISTRIBUTION

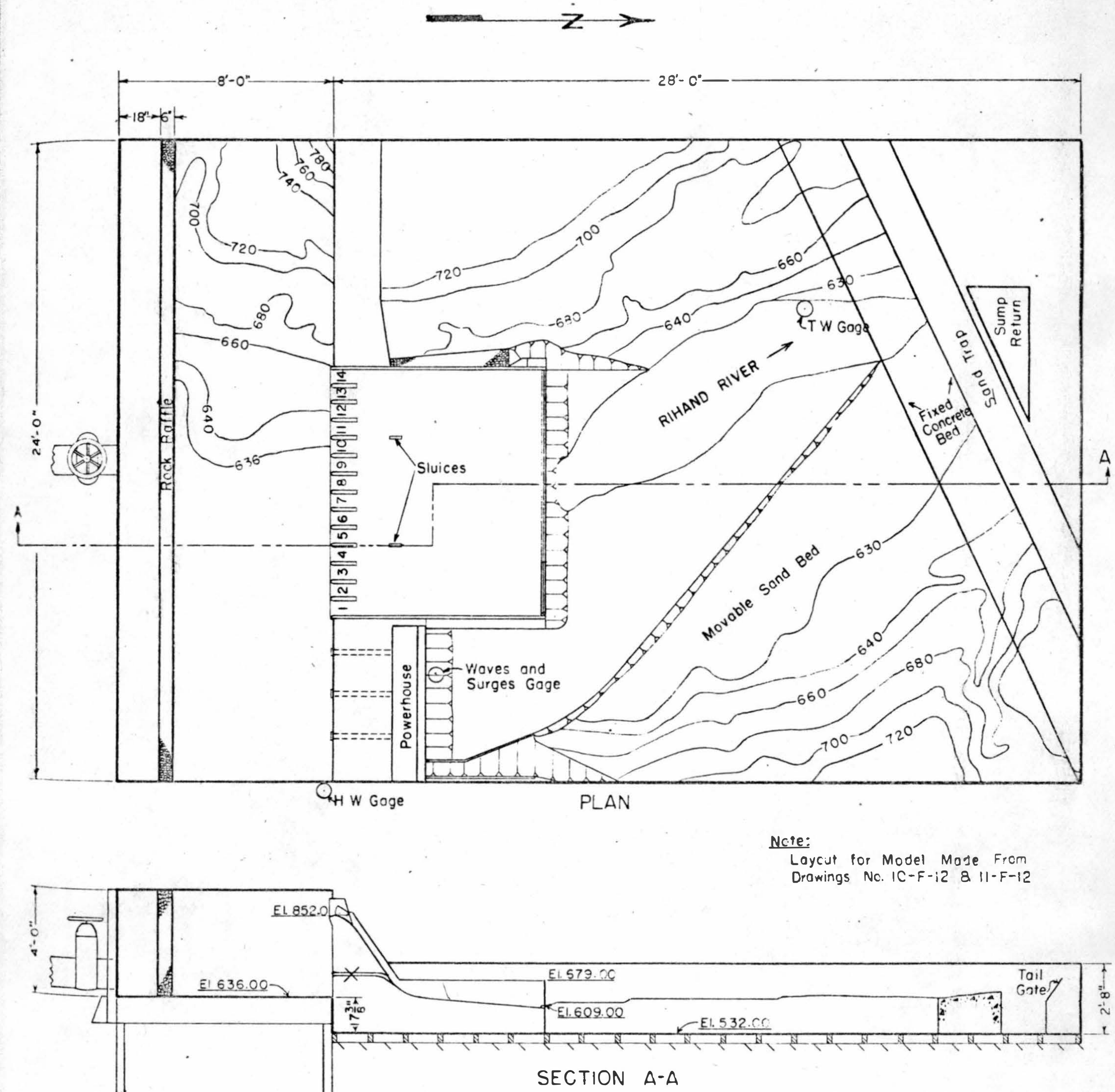
Recorded from June 1944 to July 1948
(24 hr. average except as noted)

UNITED PROVINCES, INDIA
RIHAND RIVER DEVELOPMENT

RIHAND PROJECT
AREA AND CAPACITY, DISCHARGE RATINGS, AND FREQUENCY CURVES

INTERNATIONAL ENGINEERING CO., INC.

DESIGNED GJB AHE AJC	CHK'D. AJC HMB	SUBMITTED
DRAWN GJB	INSP. LDD	RECOMMENDED
TRACED		APPROVED
10-F-11	SAN FRANCISCO, CAL. DATE Oct 19, 1949	512-A-11

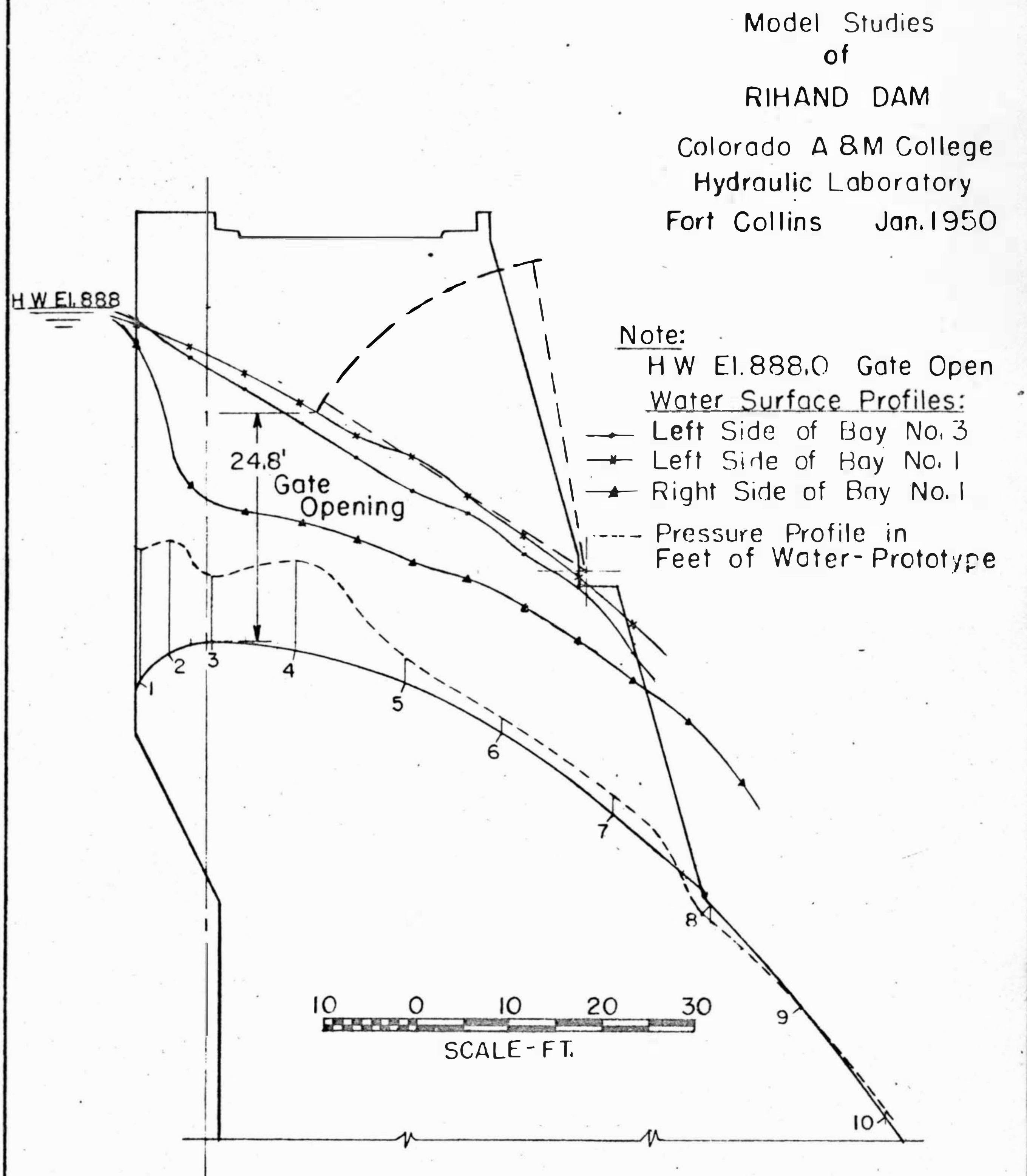


Note:
Layout for Model Made From Drawings No. 10-F-12 & 11-F-12

General Plan & Elevation
1:72 Complete Model

Model Studies
of
RIHAND DAM

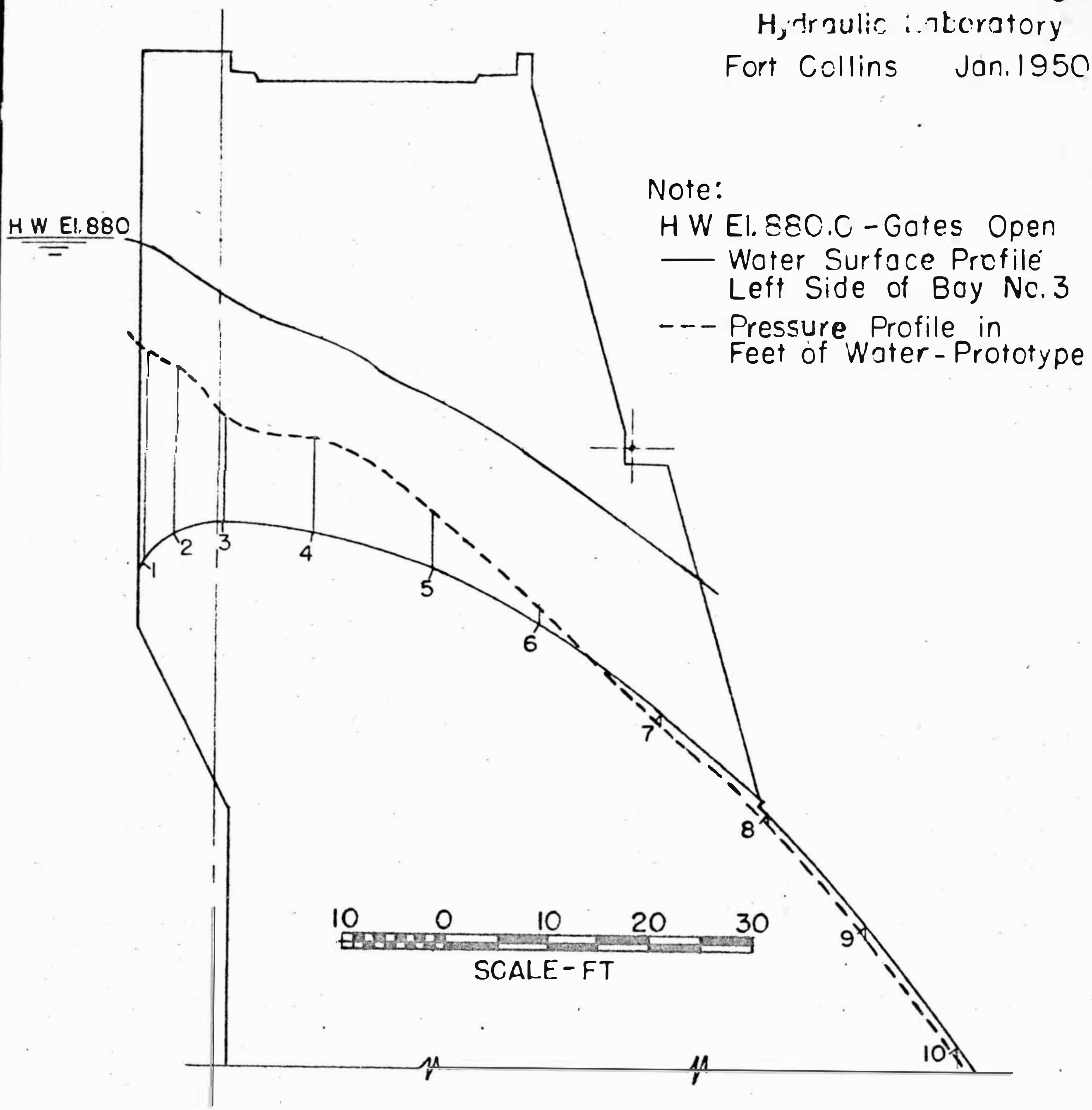
Colorado A & M College
Hydraulic Laboratory
Fort Collins Jan 1950



Pressure Distribution & Water-Surface Profiles
Crest-Overfall Spillway
H W El. 888.0 - Gates Open

Model Studies
of
RIHAND DAM

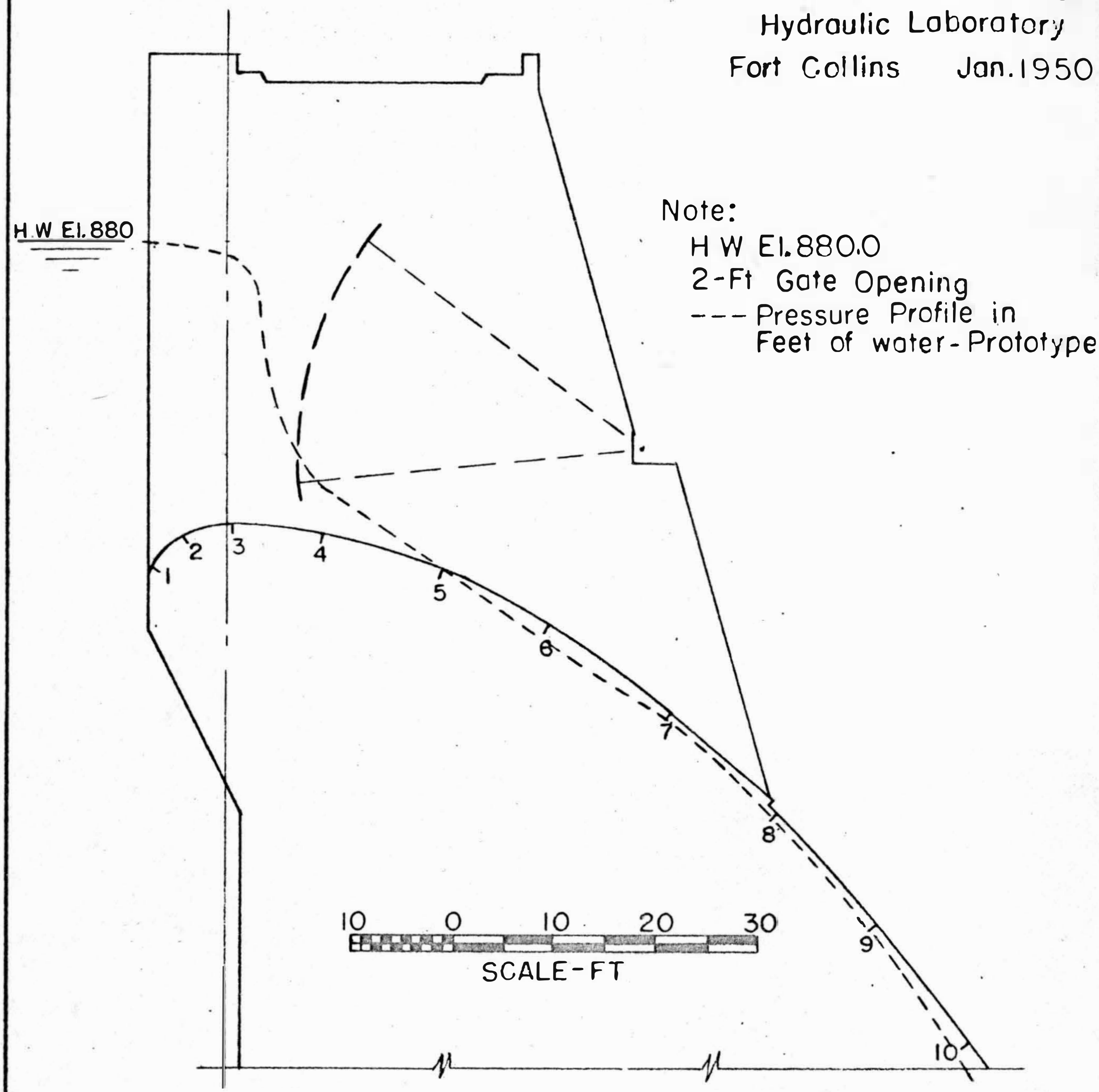
Colorado A & M College
Hydraulic Laboratory
Fort Collins Jan. 1950



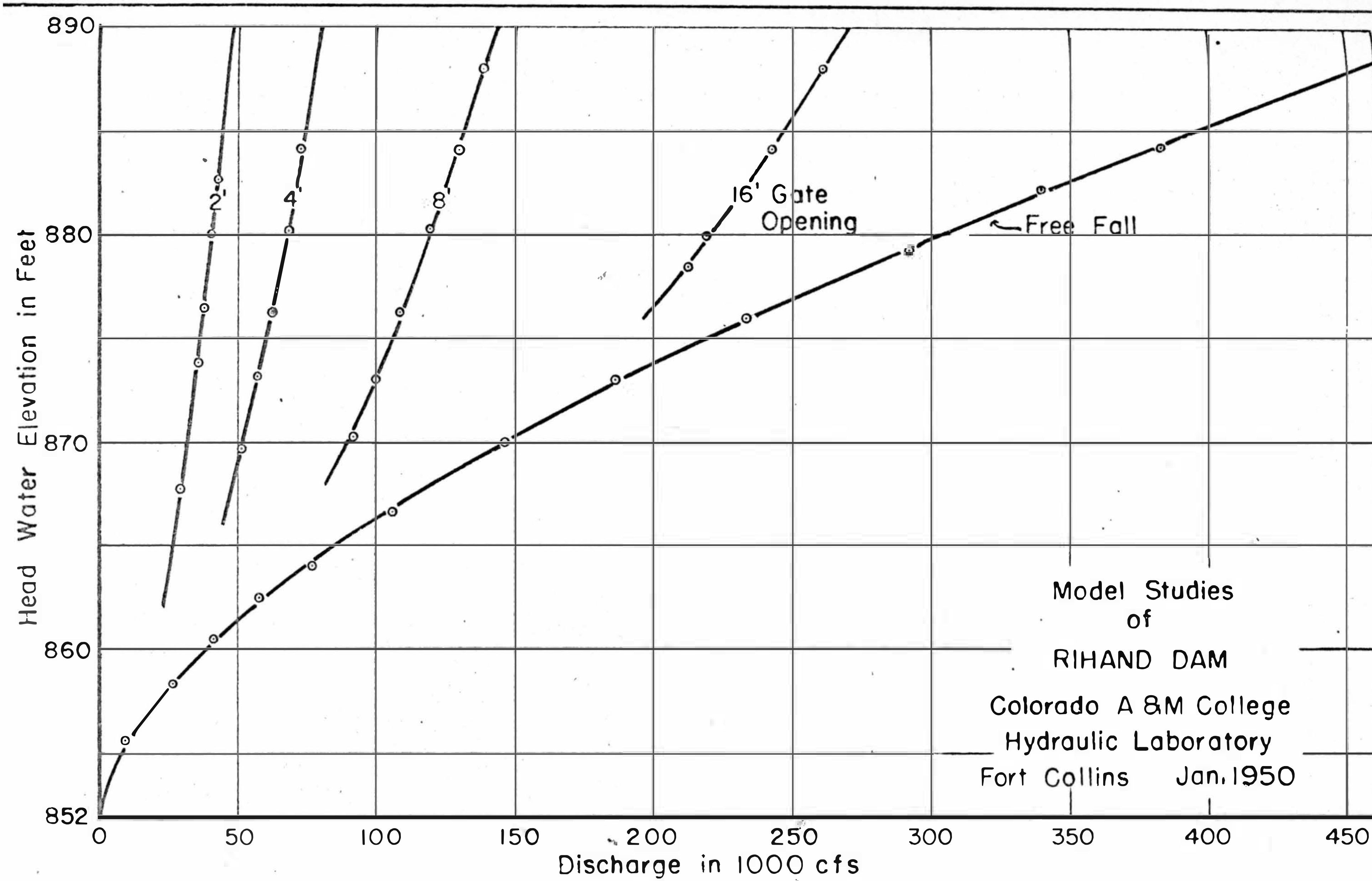
Pressure Distribution & Water-Surface Profile
Crest - Overfall Spillway
H W El. 880.0 - Gates Open

Model Studies
of
RIHAND DAM

Colorado A & M College
Hydraulic Laboratory
Fort Collins Jan. 1950

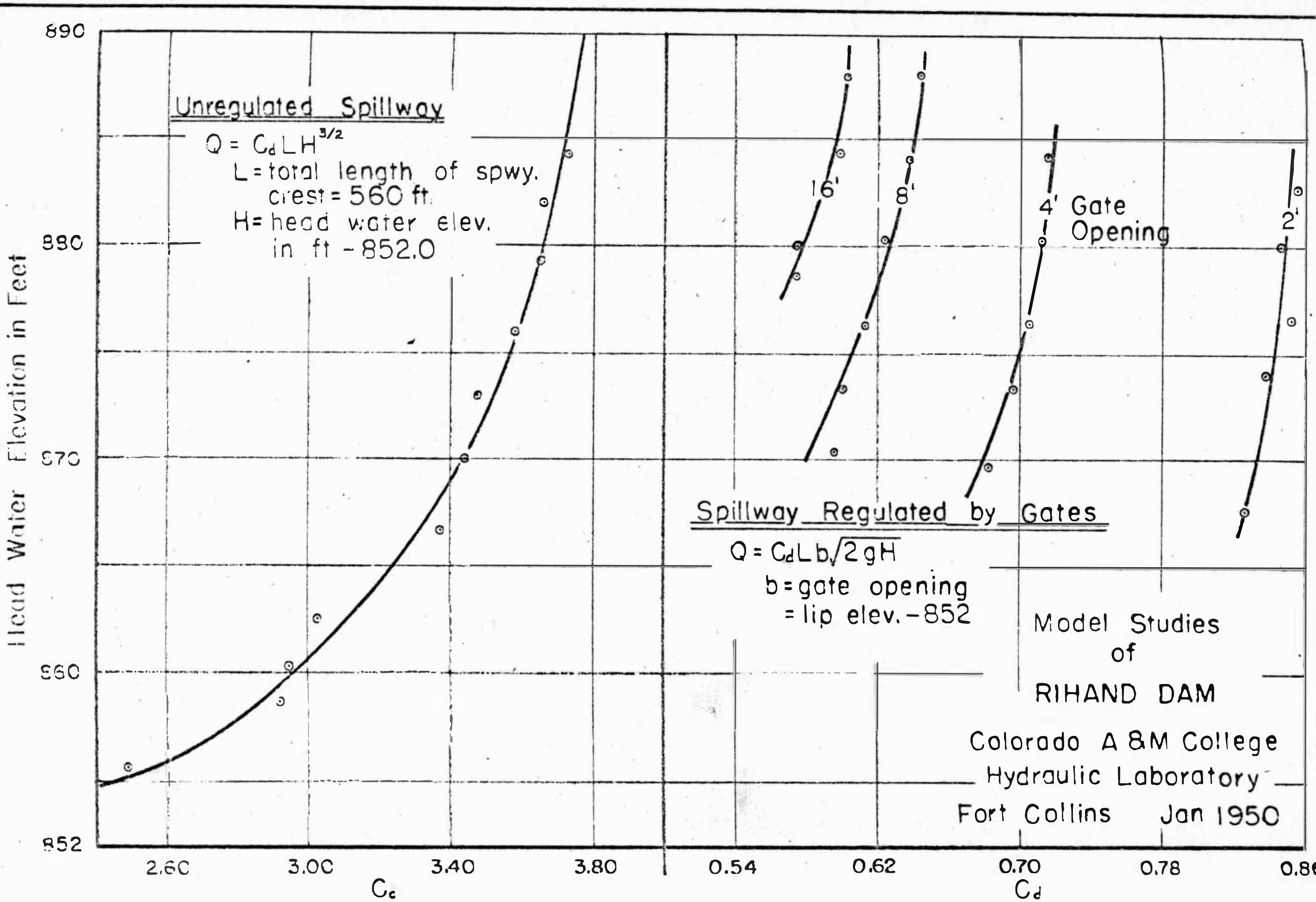


Pressure Distribution
Crest – Overfall Spillway
H W El. 880.0 – 2-Ft. Gate Opening



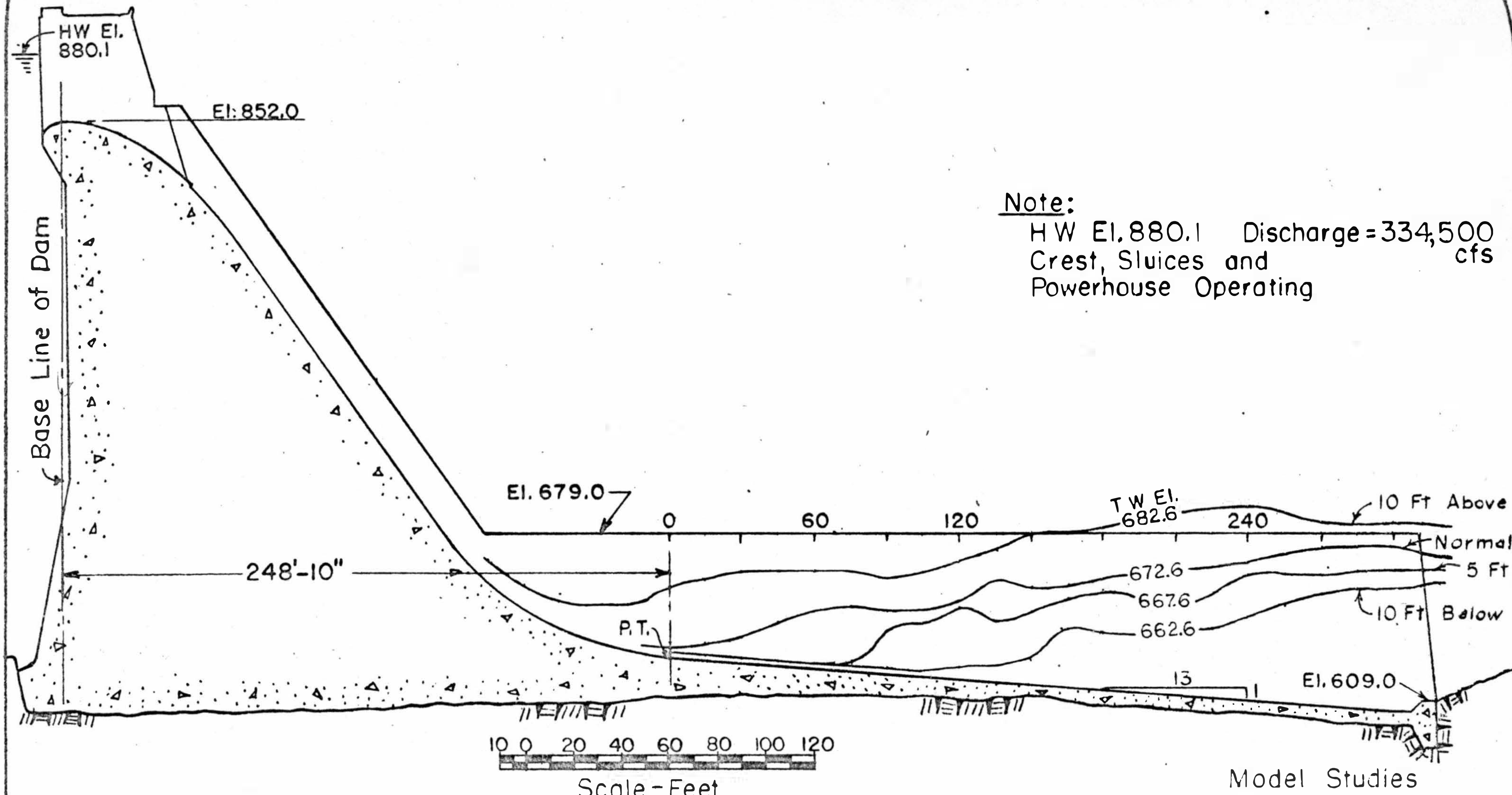
Variation of Discharge with Head
Overfall Spillway

FIGURE 15



Variation of Discharge Coefficient with Head
Overfall Spillway

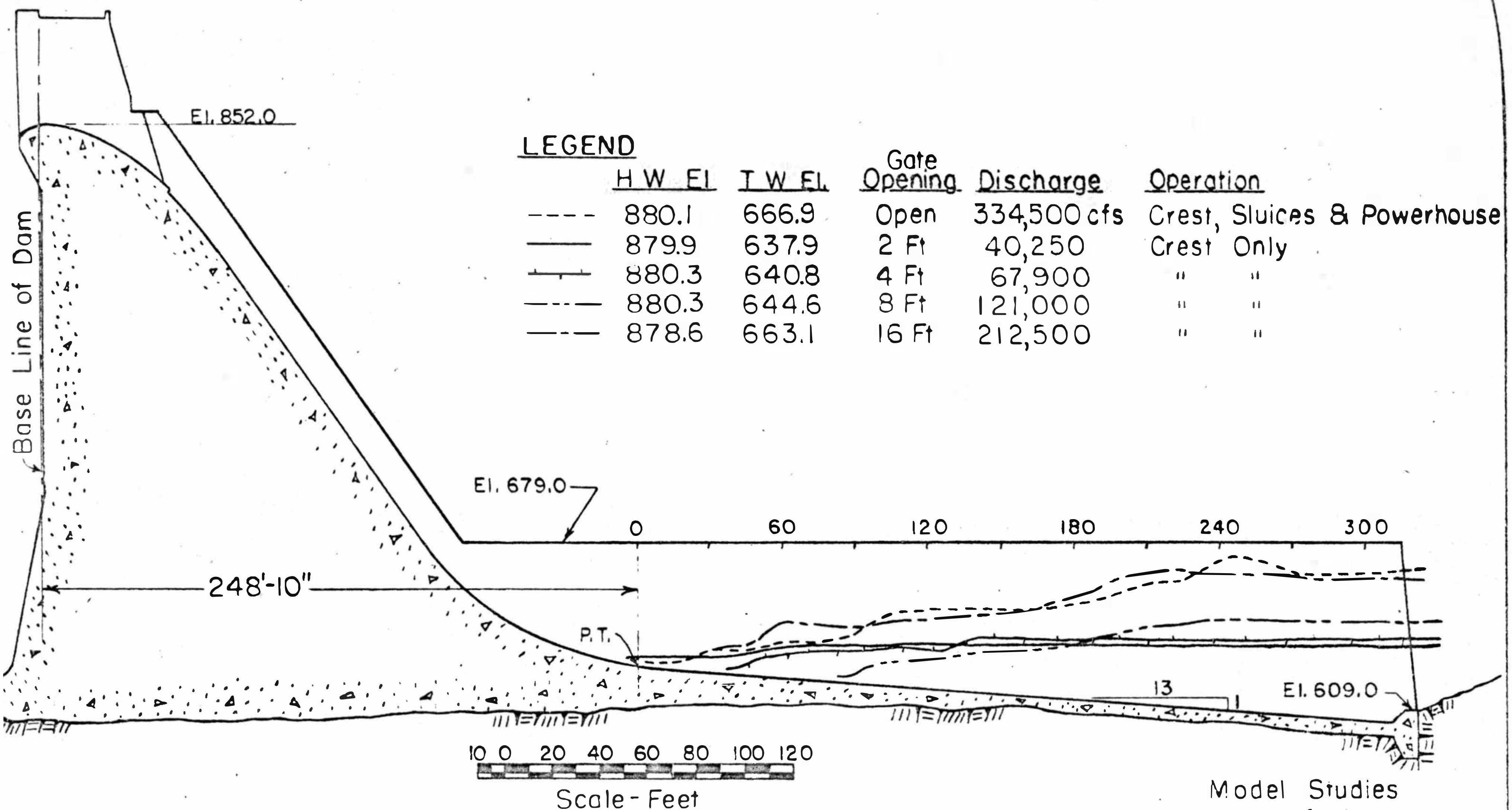
FIGURE 16



Water Surface Profiles
Stilling Basin - Overfall Spillway
Varying Tailwater

Model Studies of RIHAND DAM
Colorado A & M College
Hydraulic Laboratory
Fort Collins Jan. 1950

FIGURE 17



Water Surface Profiles
Stilling Basin – Overfall Spillway
Varying Discharge

Model Studies
of
RIHAND DAM

Colorado A&M College
Hydraulic Laboratory
Fort Collins Jan. 1950

FIGURE 18

PLATE 1
Construction Details - Head Box and Tail Box
Plywood Wall Panels being Placed
1:72 Complete Model

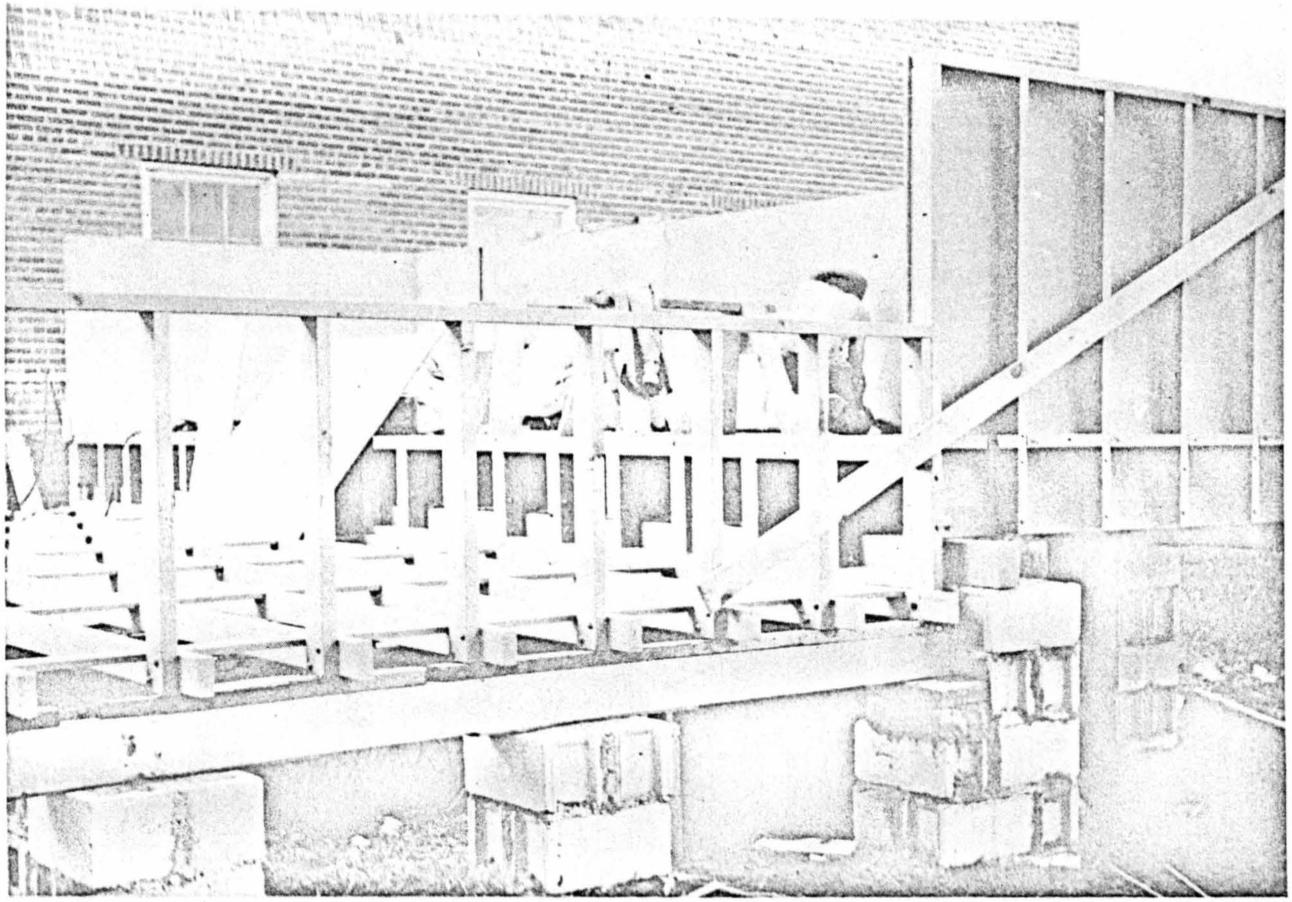


PLATE 2

Construction Details - Head Box and Tail Box
Supporting Columns and Framing Details

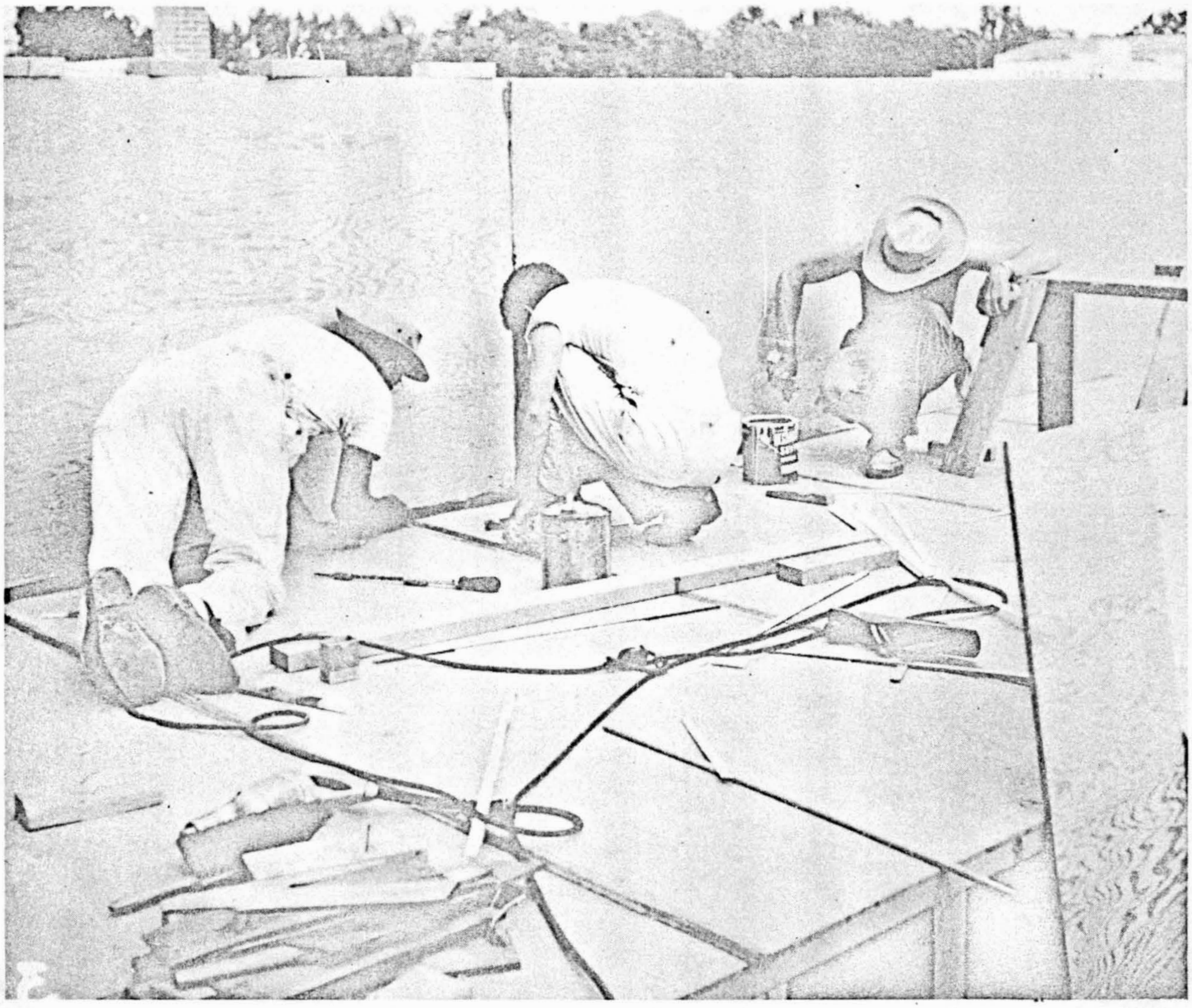


PLATE 3
Construction Details - Head Box
Joint Construction with Waterproof Rubber Seal

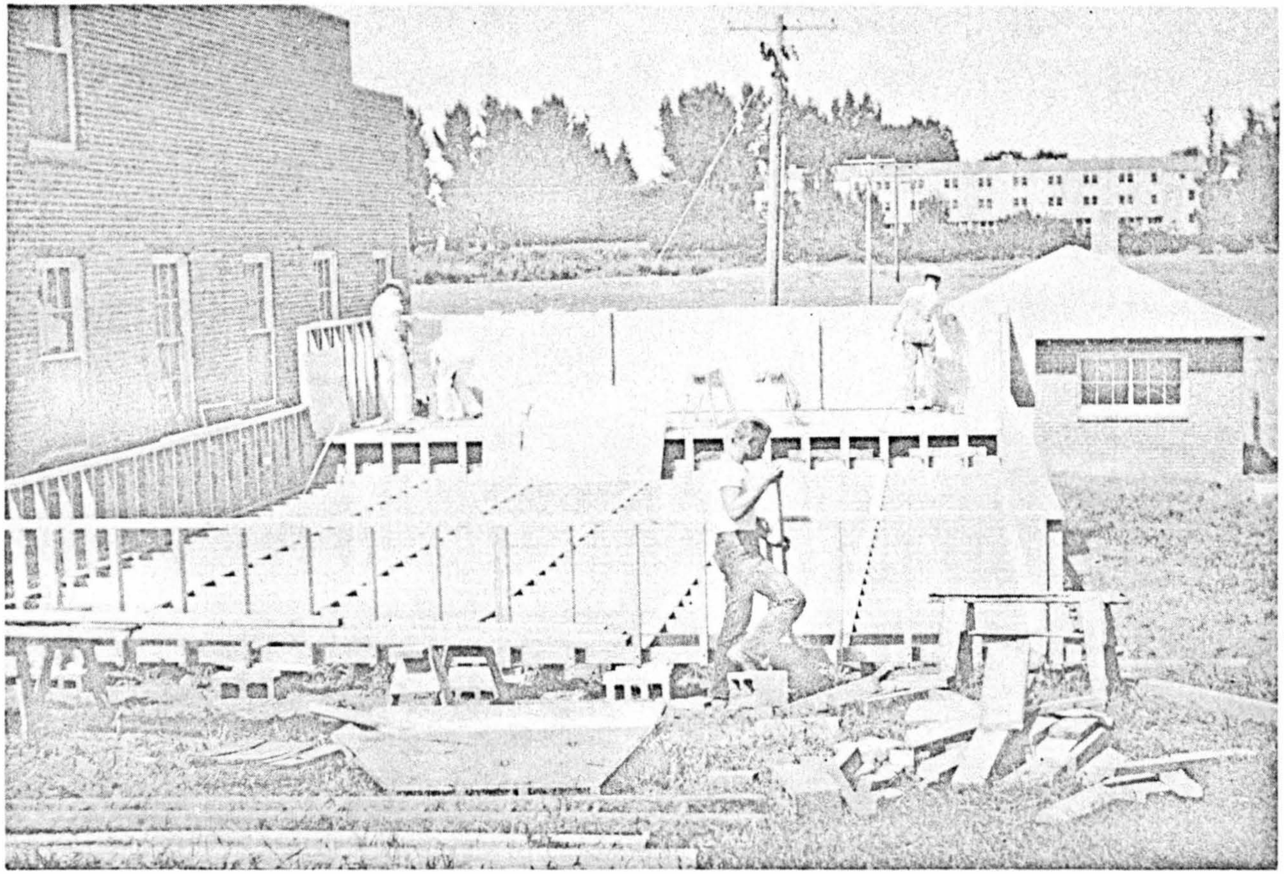


PLATE 4
Construction Details - Head Box and Tail Box
General View

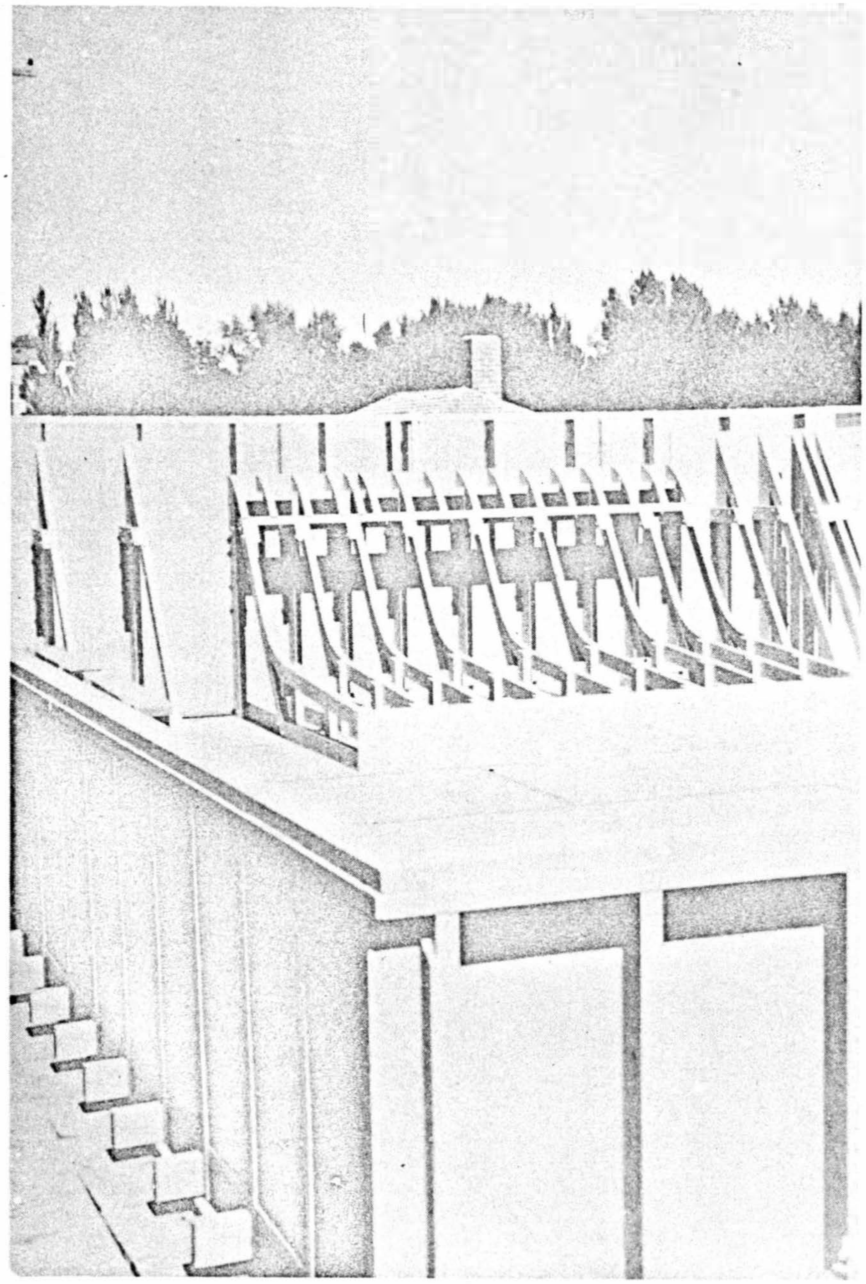


PLATE 5
Construction Details - Overfall Spillway
General View of Framework

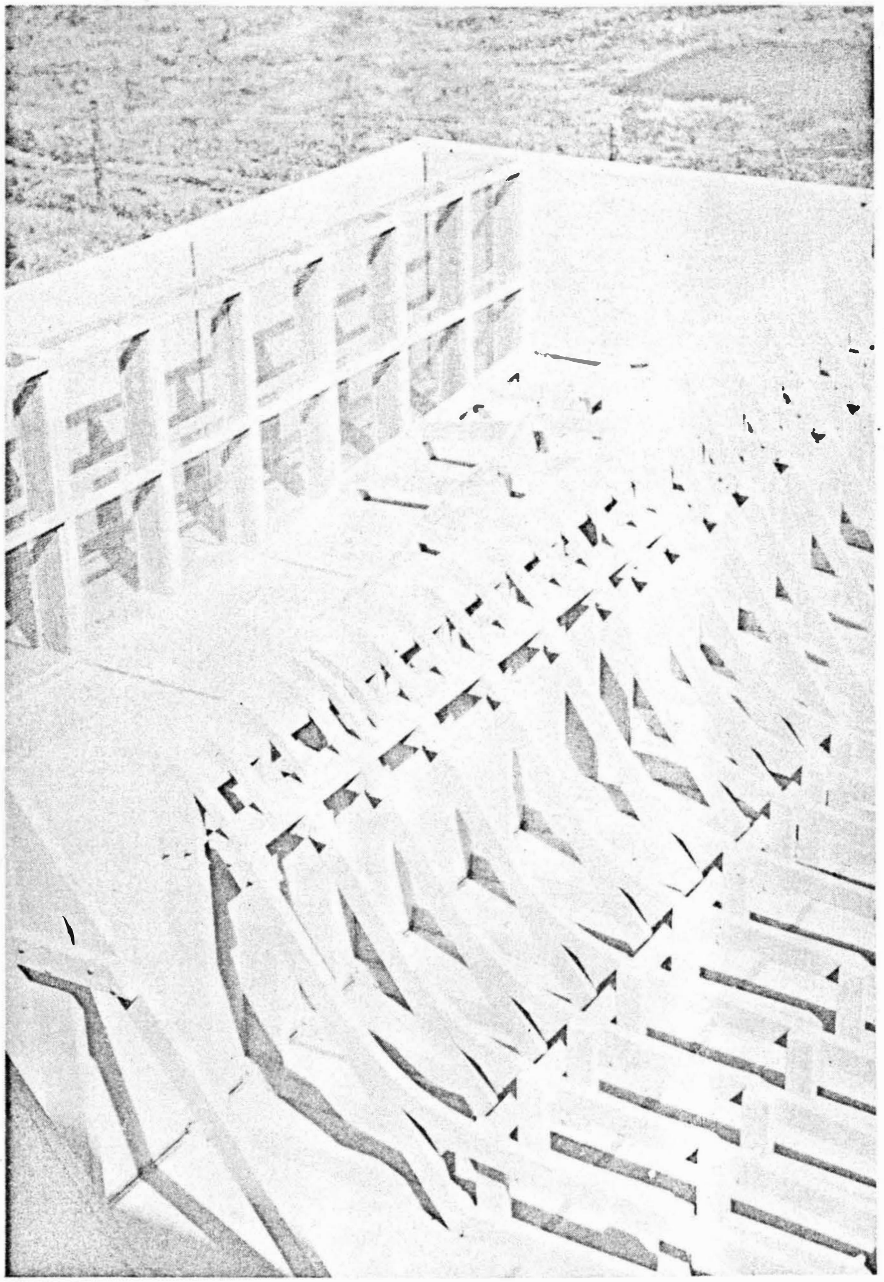


PLATE 6
Construction Details - Head Box and Overfall Spillway
Frame and Wire Screen of Rock Baffle
Templet Frames for Topography
Overfall Spillway Framework

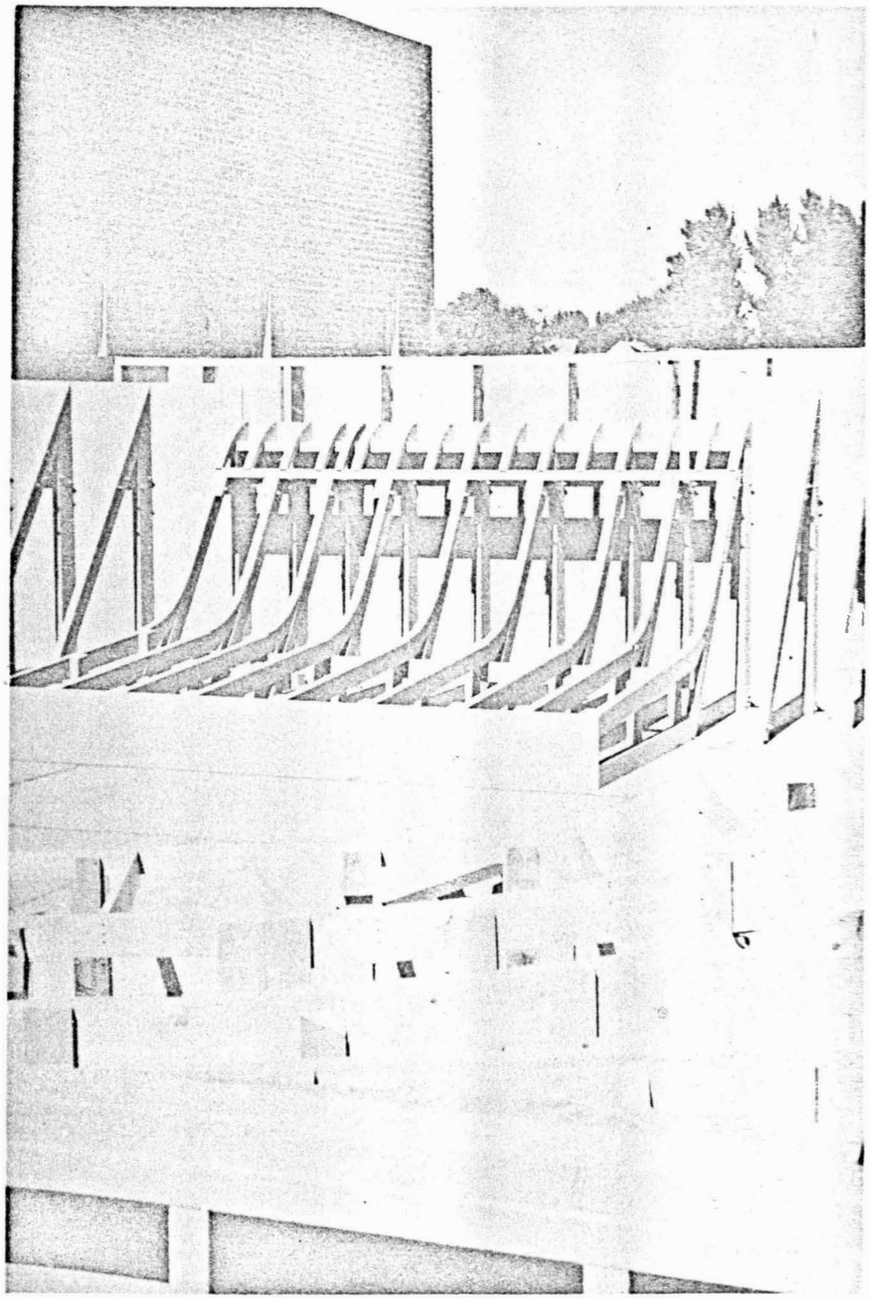


PLATE 7
Construction Details - Head Box and Overfall Spillway
Overfall Spillway Framing
Templet Frames for Topography in Foreground

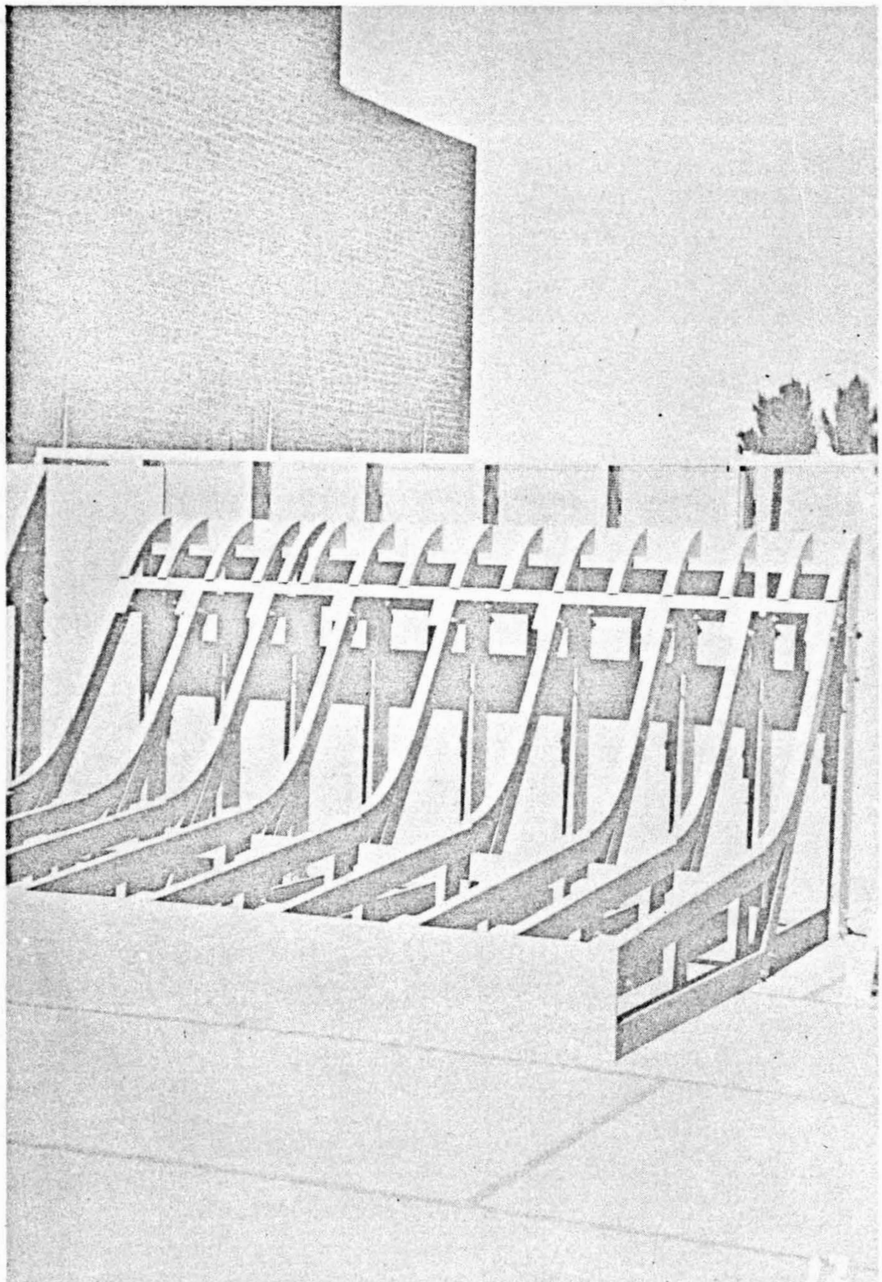


PLATE 8
Construction Details - Overfall Spillway
Closeup View of Framework

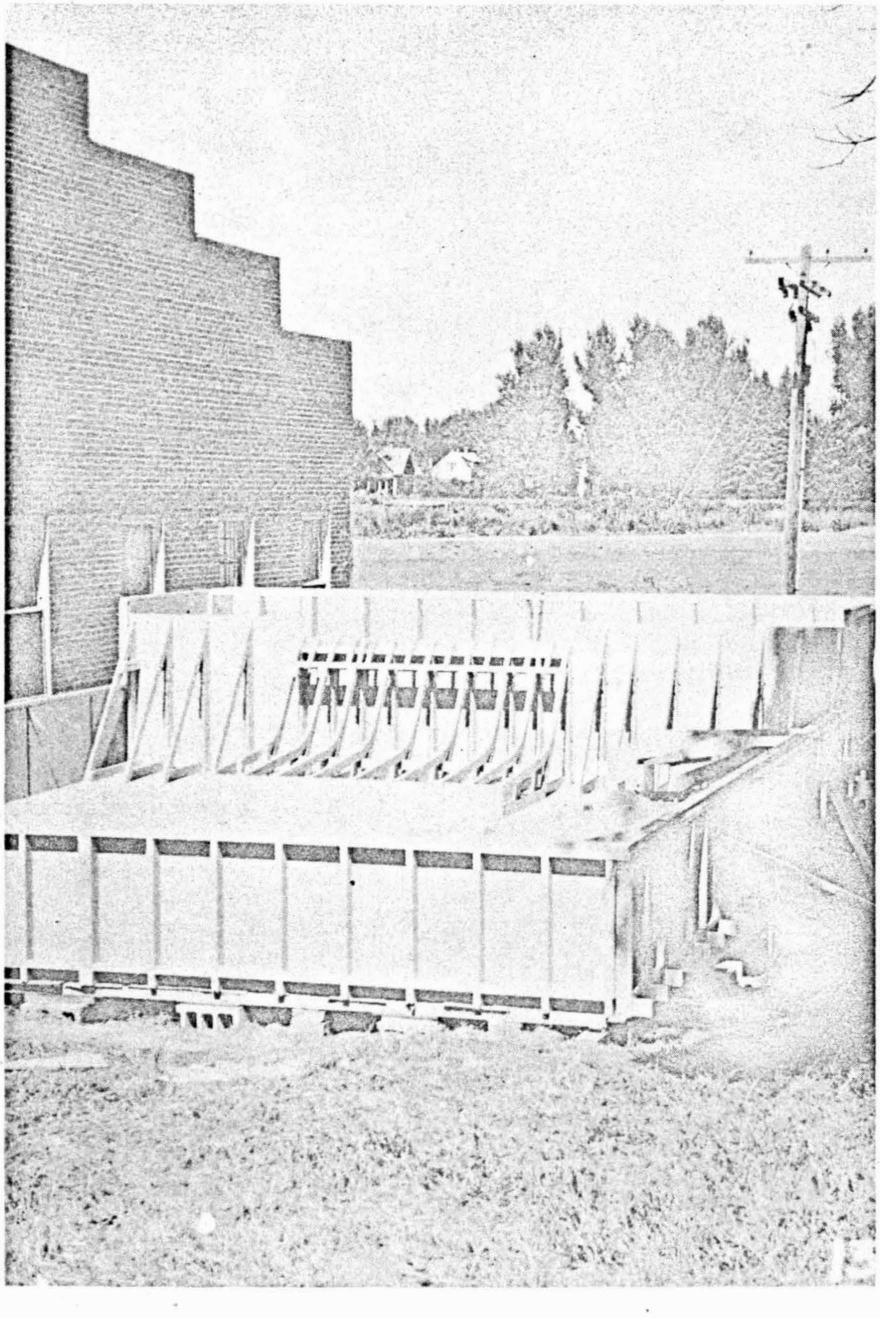


PLATE 9
Construction Details - Entire Model
General View from Downstream

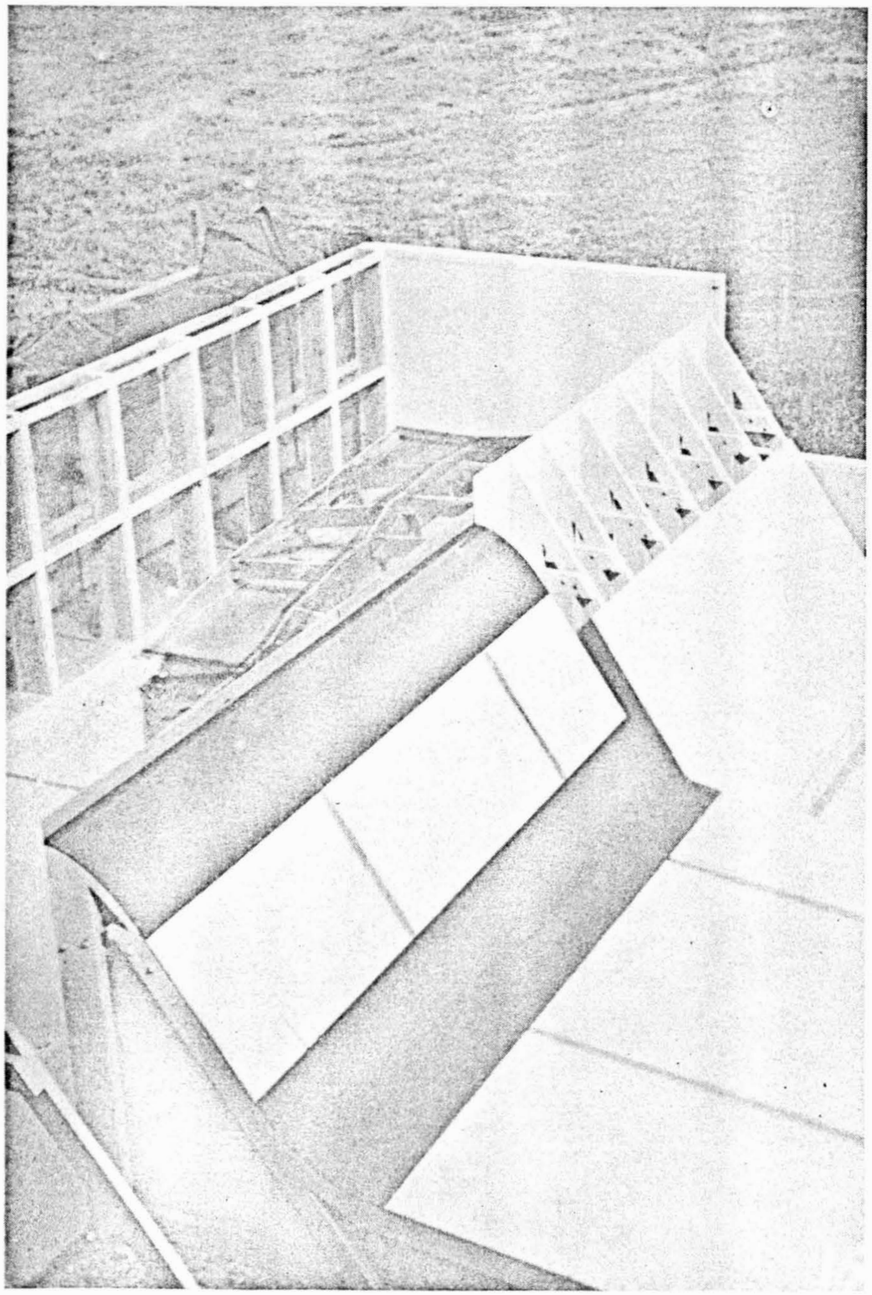


PLATE 10
Construction Details - Head Box and Overfall Spillway
Topography Templet Frames with Metal Lath Covering
Masonite Crest and Bucket
Plywood Facing of Overfall Spillway

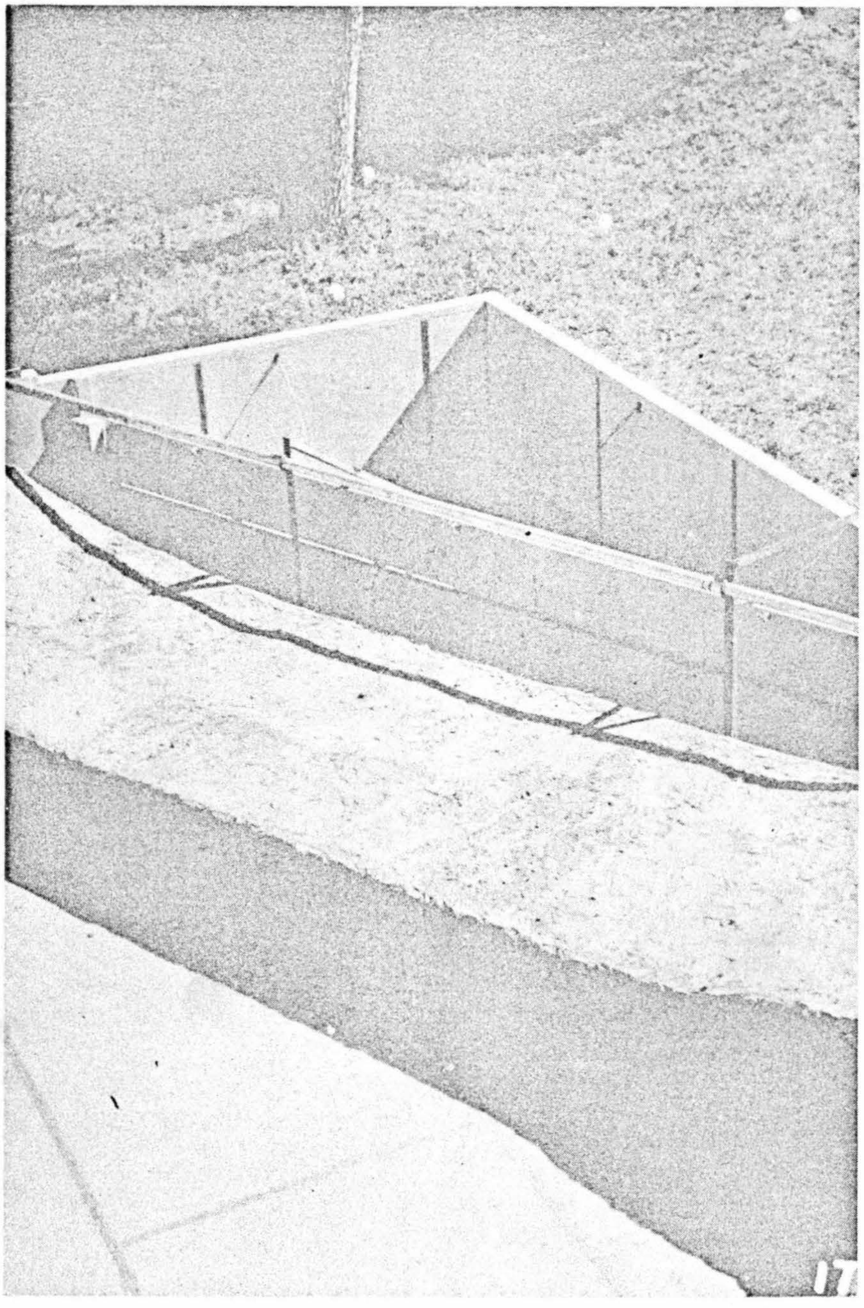


PLATE 11
Construction Details - Tail Box
Concrete Scratch Coat on Topography Framework
Sand Trap, Tail Gate, and Chute for Return Flow to Sump

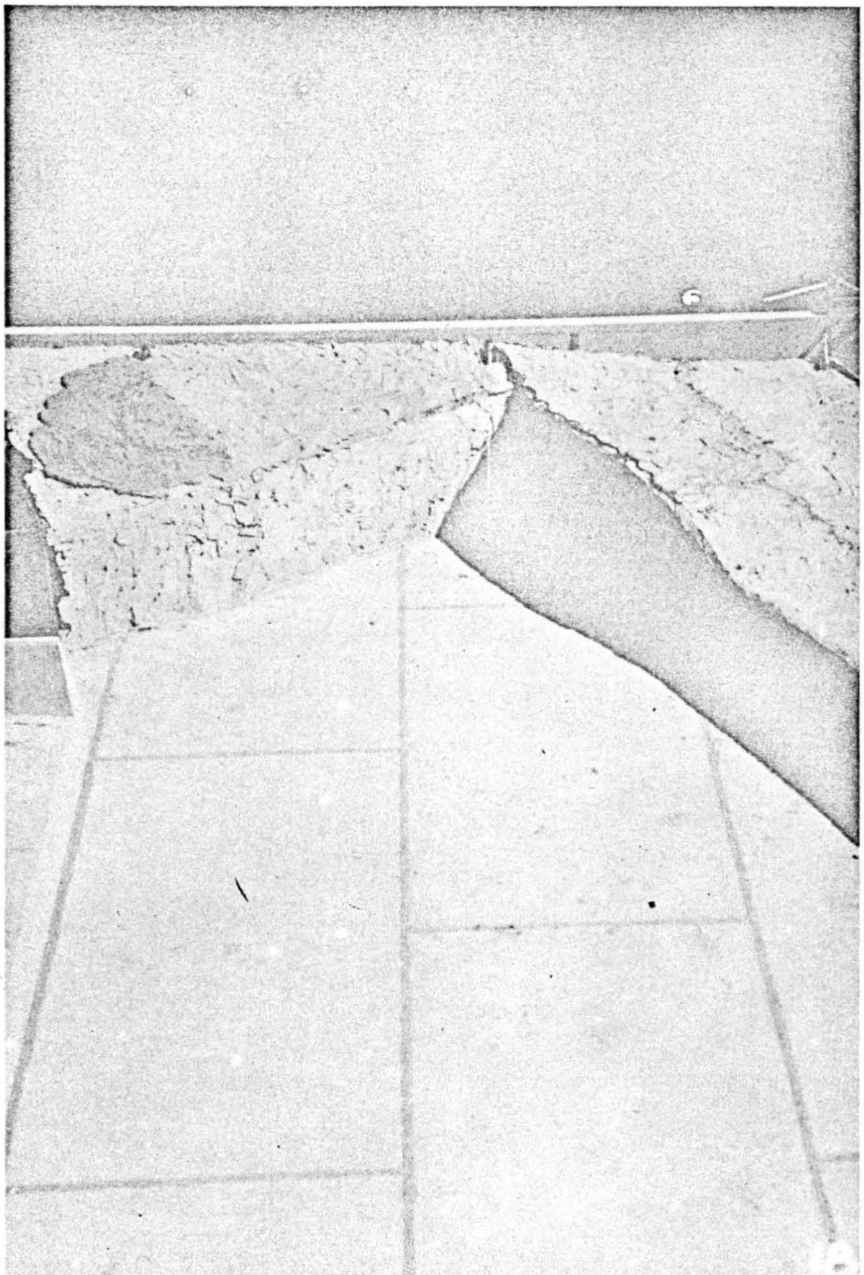


PLATE 12
Construction Details - Tail Box
Topography Templet Framing
Scratch Coat and Finish Coat Application

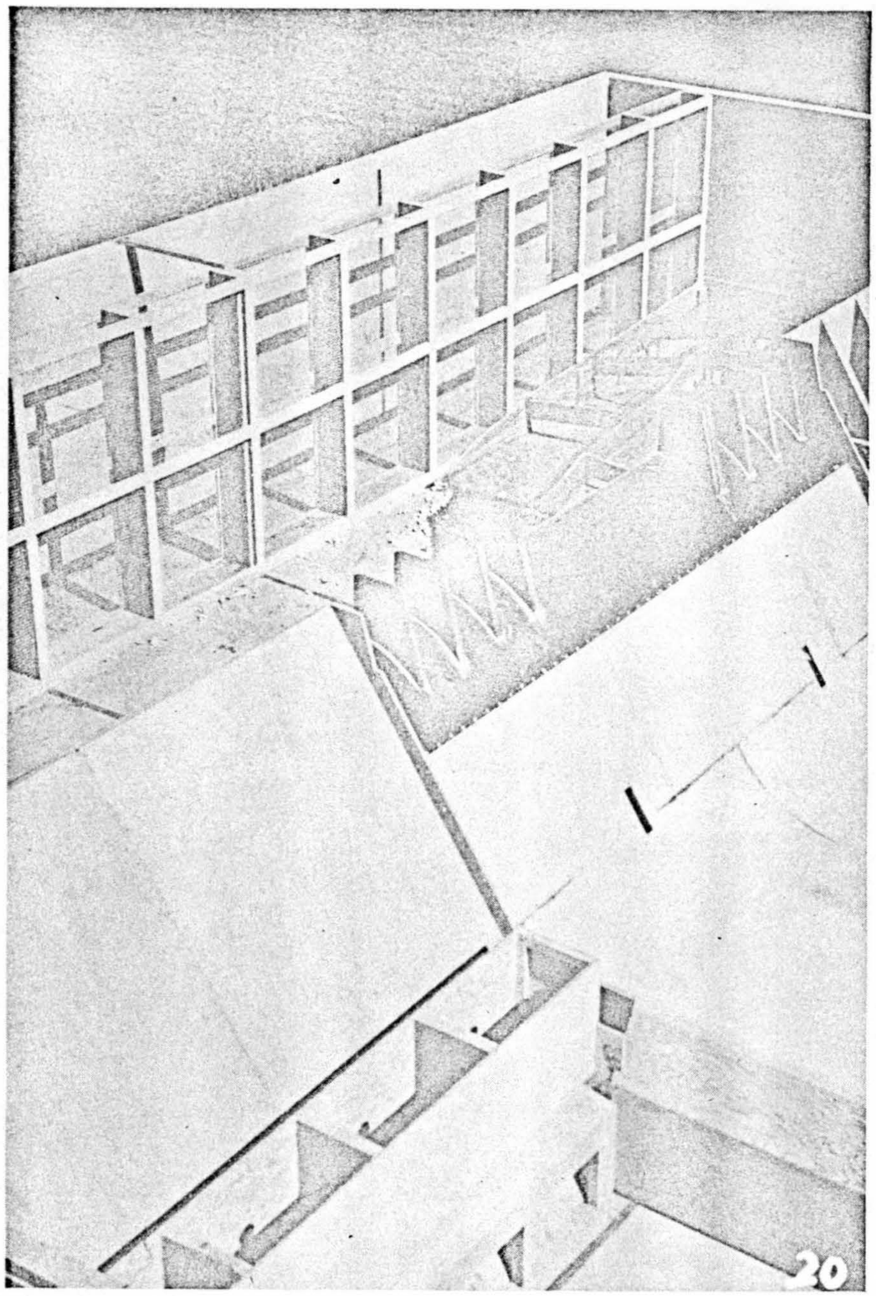


PLATE 13
Construction Details - Overfall Spillway and Powerhouse
Mahogany Piers on Crest of Spillway
Power House Adjacent to Right Training Wall

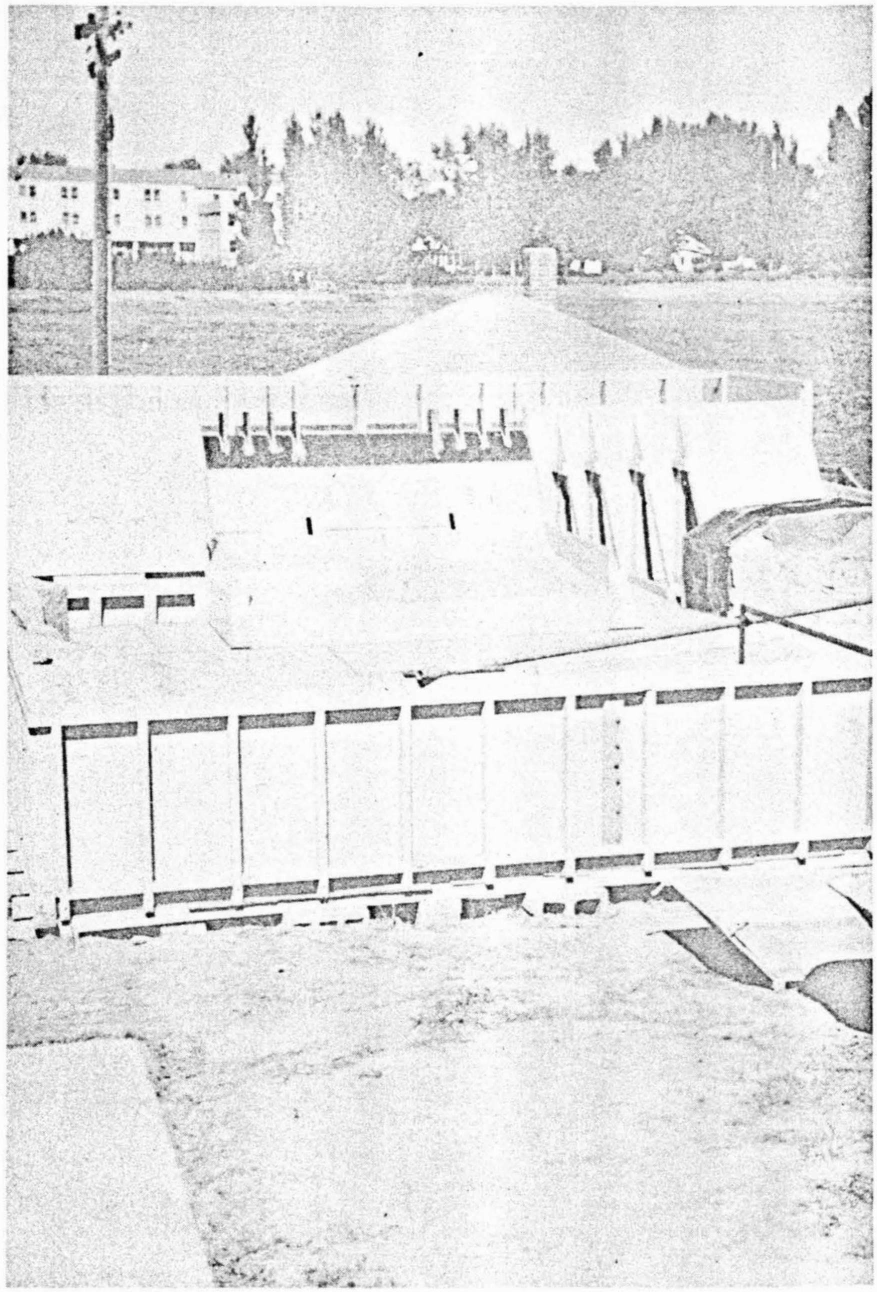


PLATE 14
Construction Details - Entire Model
General View from Downstream
Spillway Piers and Covering in Place

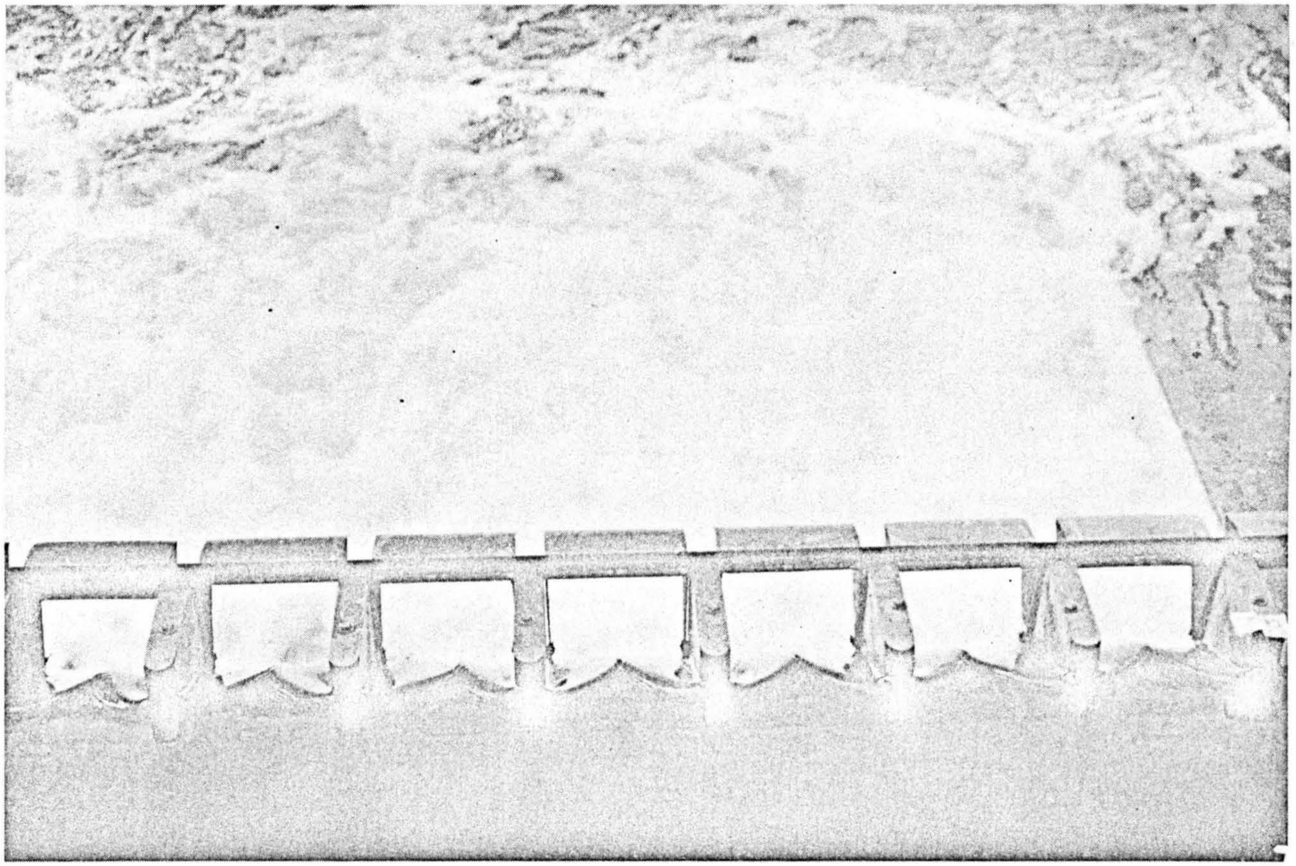


PLATE 15
Flow Patterns - Overfall Spillway
Approach and Drawdown at Piers - View from Upstream
HW Elev 888.1, - Q = 455,000 cfs - Spillway Flow Only

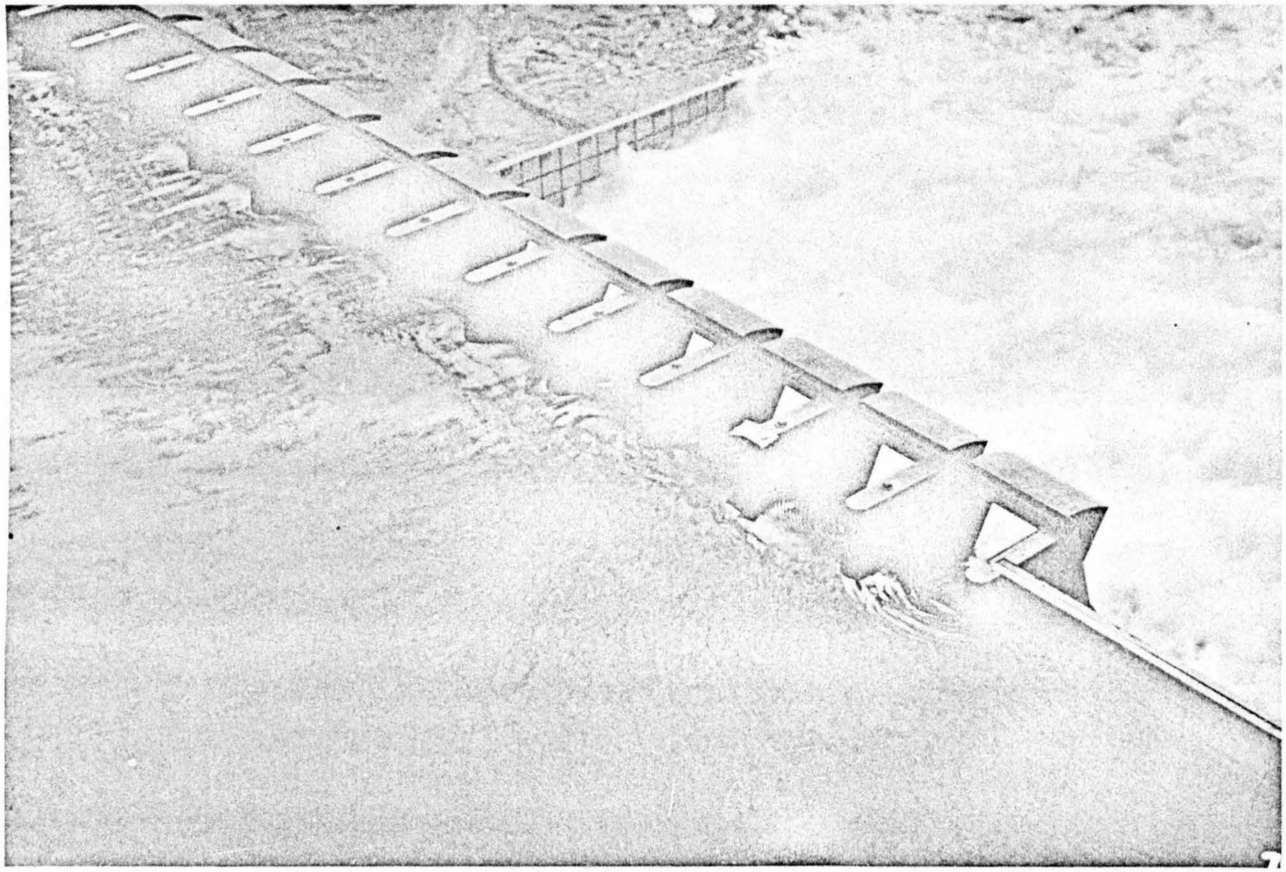


PLATE 16
Flow Patterns - Overfall Spillway
Approach and Drawdown at Piers - Side View
HW Elev 888.1 - Q = 455,000 cfs, Spillway Flow Only

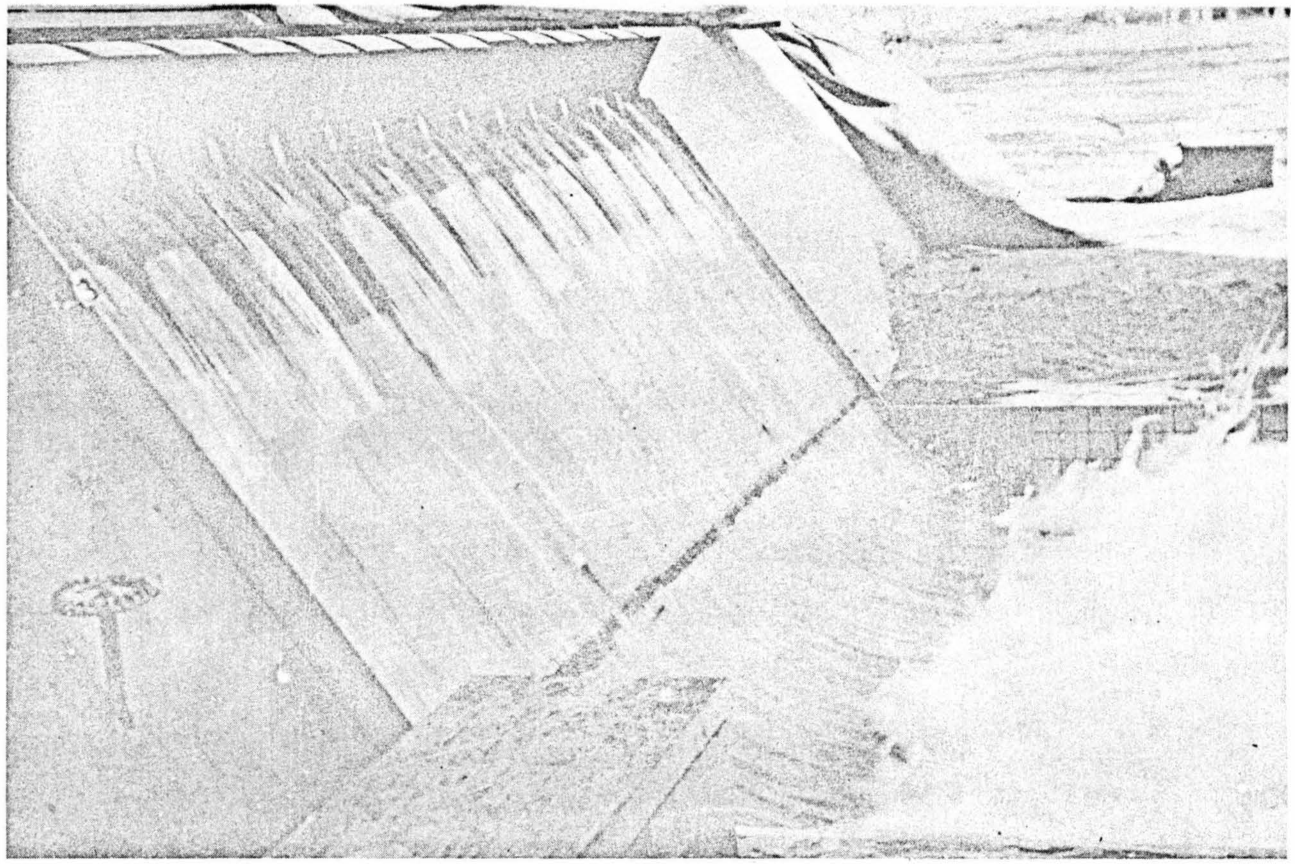


PLATE 17
Flow Patterns - Overfall Spillway
Spillway Training Walls Overtopped by Flow
HW Elev 887.6 - TW Elev 679.7, Q = 477,000 cfs

PLATE 18
Flow Patterns - Overfall Spillway
HW Elev 888.1, Q = 455,000 cfs, Spillway Flow Only

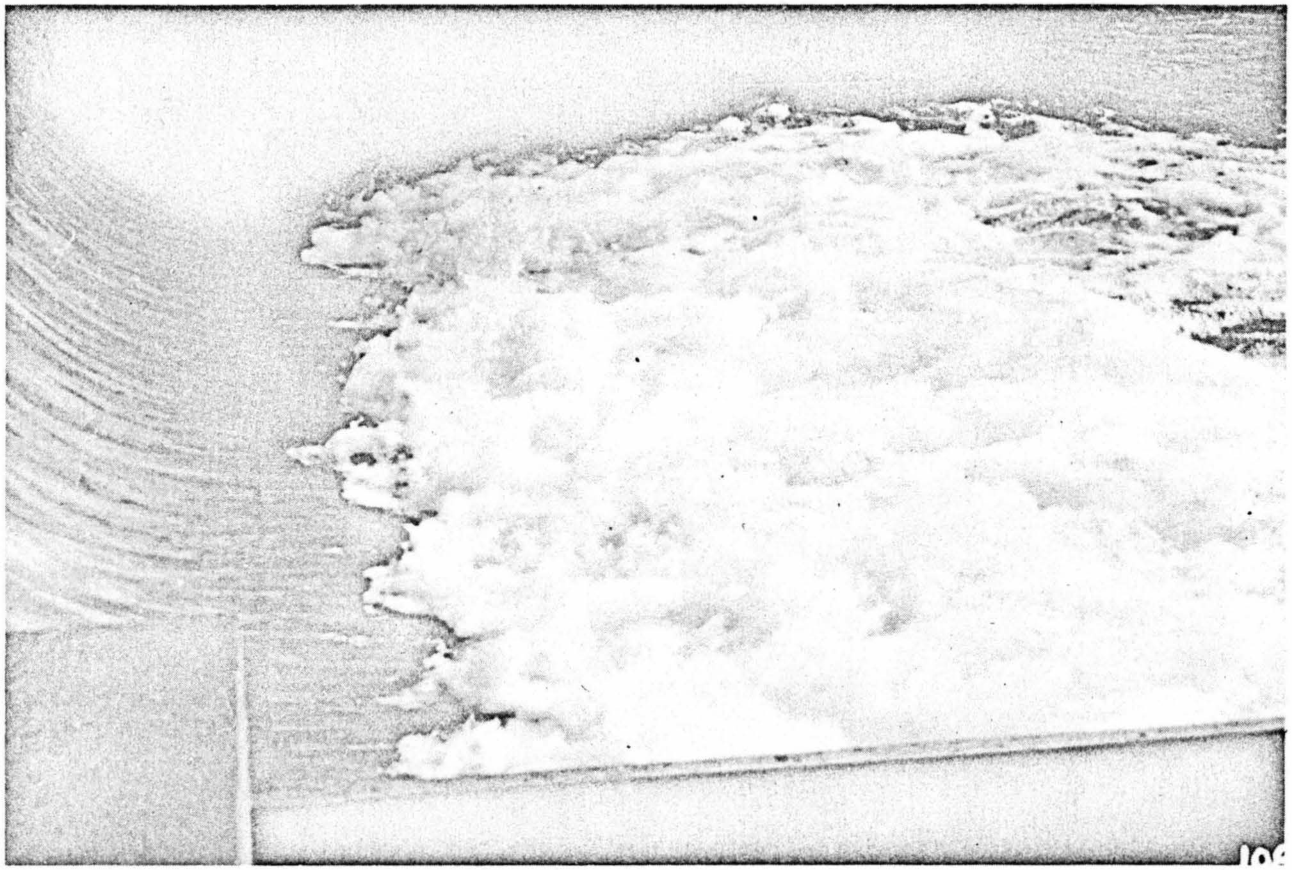


PLATE 19

Water Surface Profiles - Stilling Basin

Tailwater 6.1 Feet Below Normal

HW Elev 880.1, TW Elev 666.5, Q = 334,500 cfs

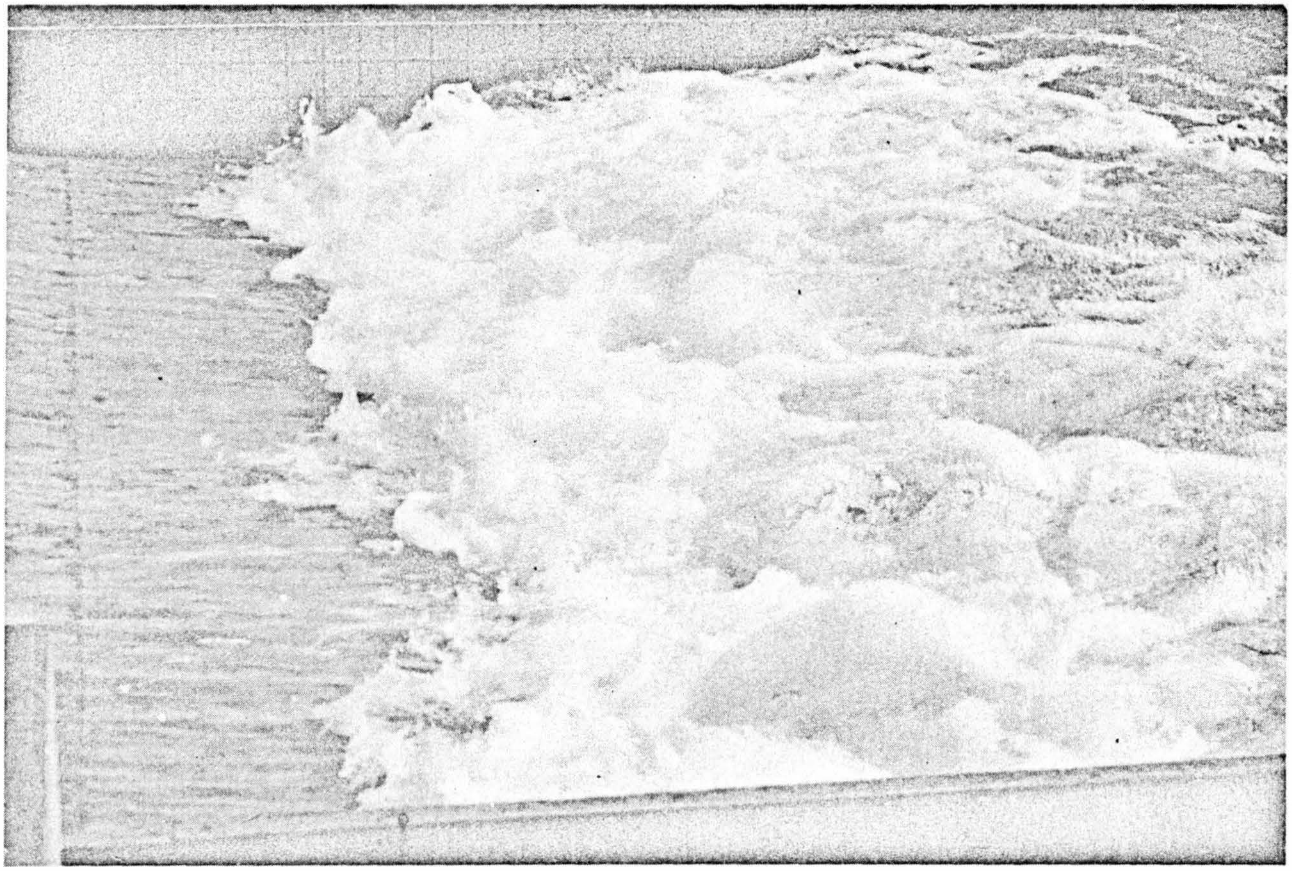


PLATE 20
Water Surface Profiles - Stilling Basin
Tail Water 11.1 Feet Below Normal
HW Elev 880.1, TW Elev 661.5, Q = 334,500 cfs

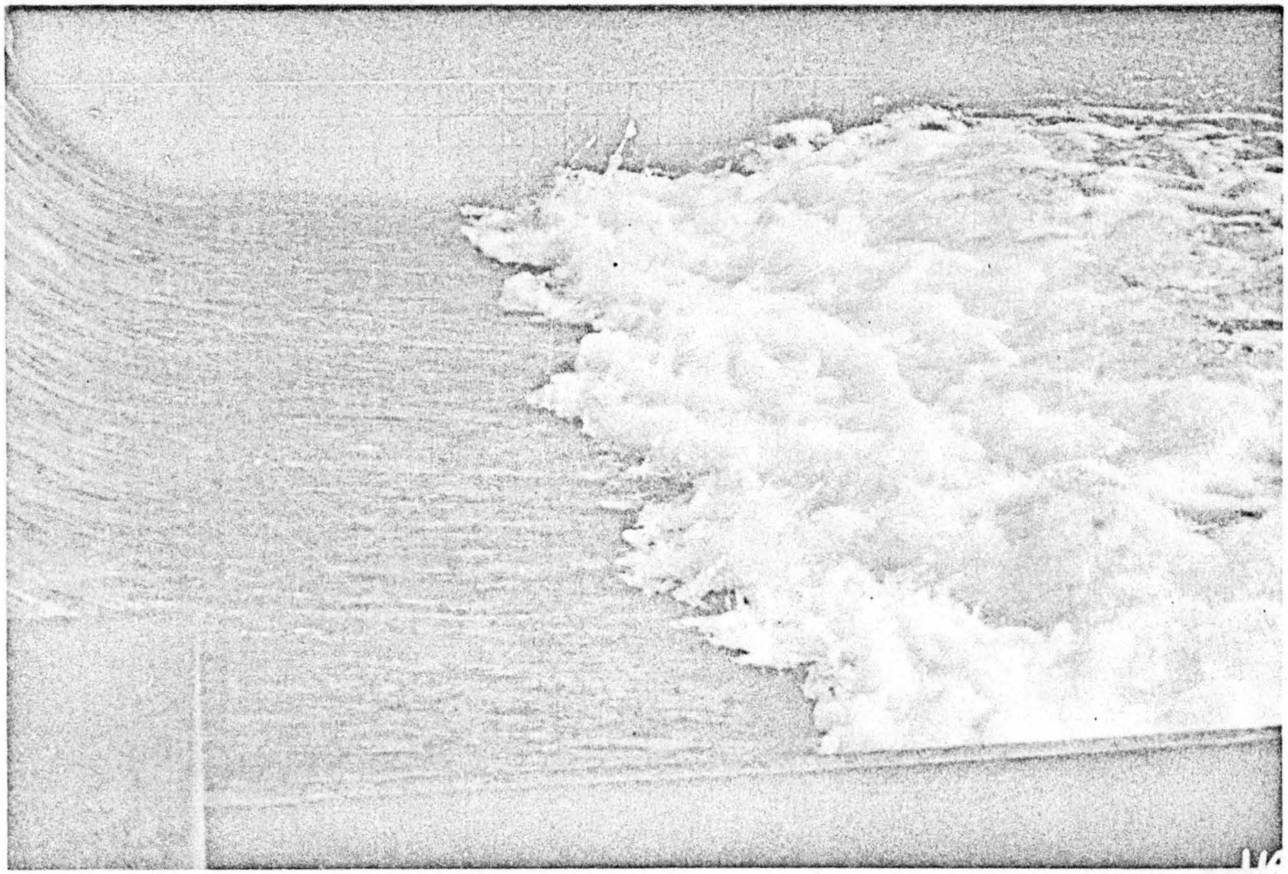


PLATE 21
Water Surface Profiles - Stilling Basin
Tail Water 16.1 Feet Below Normal
HW Elev 880.1, TW Elev 656.5, Q = 334,500 cfs

PLATE 22
Water Surface Profiles - Stilling Basin
Tail Water 3.9 Feet Above Normal
HW Elev 880.1, TW Elev 676.5, Q = 334,500 cfs

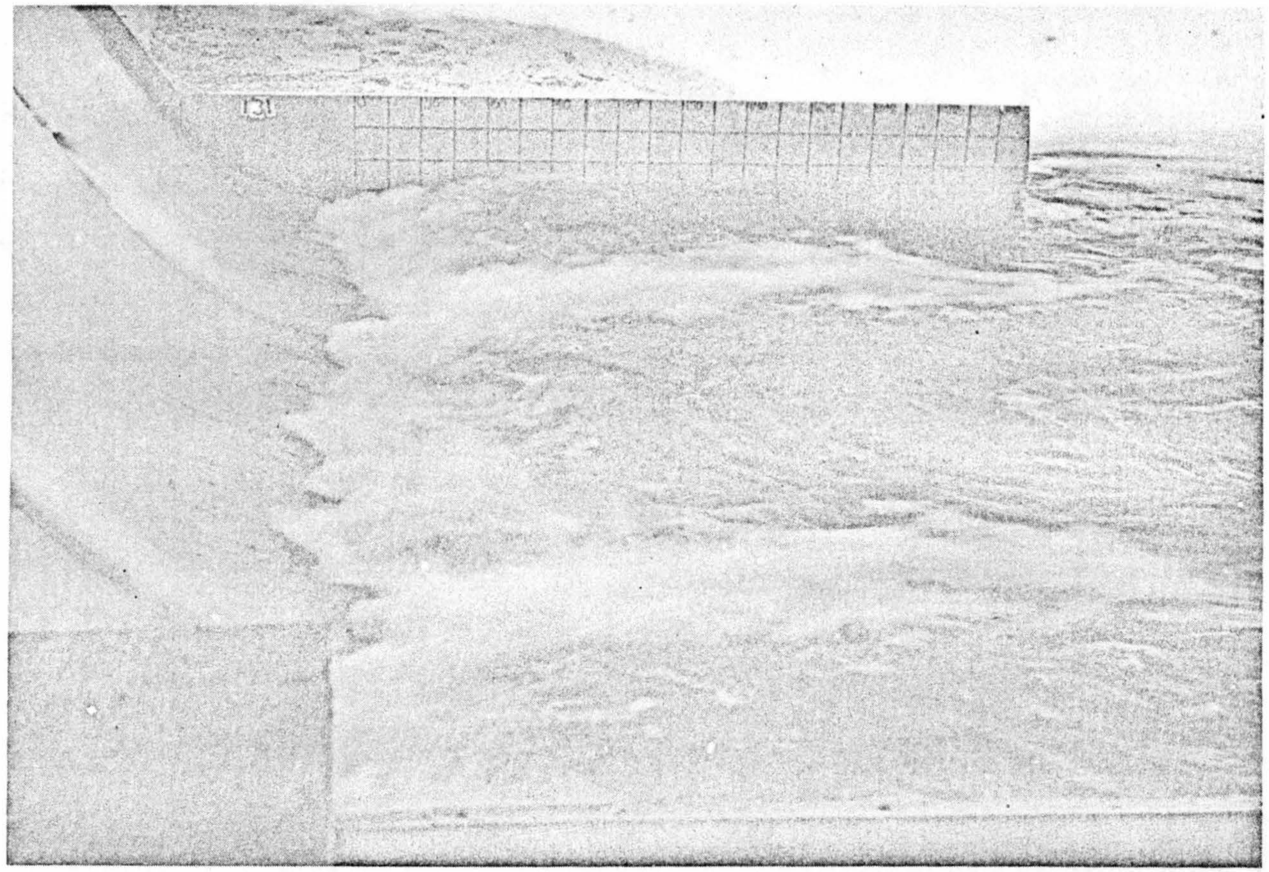


PLATE 23
Flow Patterns - Stilling Basin
Effect of Sluice Operation on Stilling Basin Performance
HW Elev 880.5, TW Elev 664, Q = 69,500 cfs

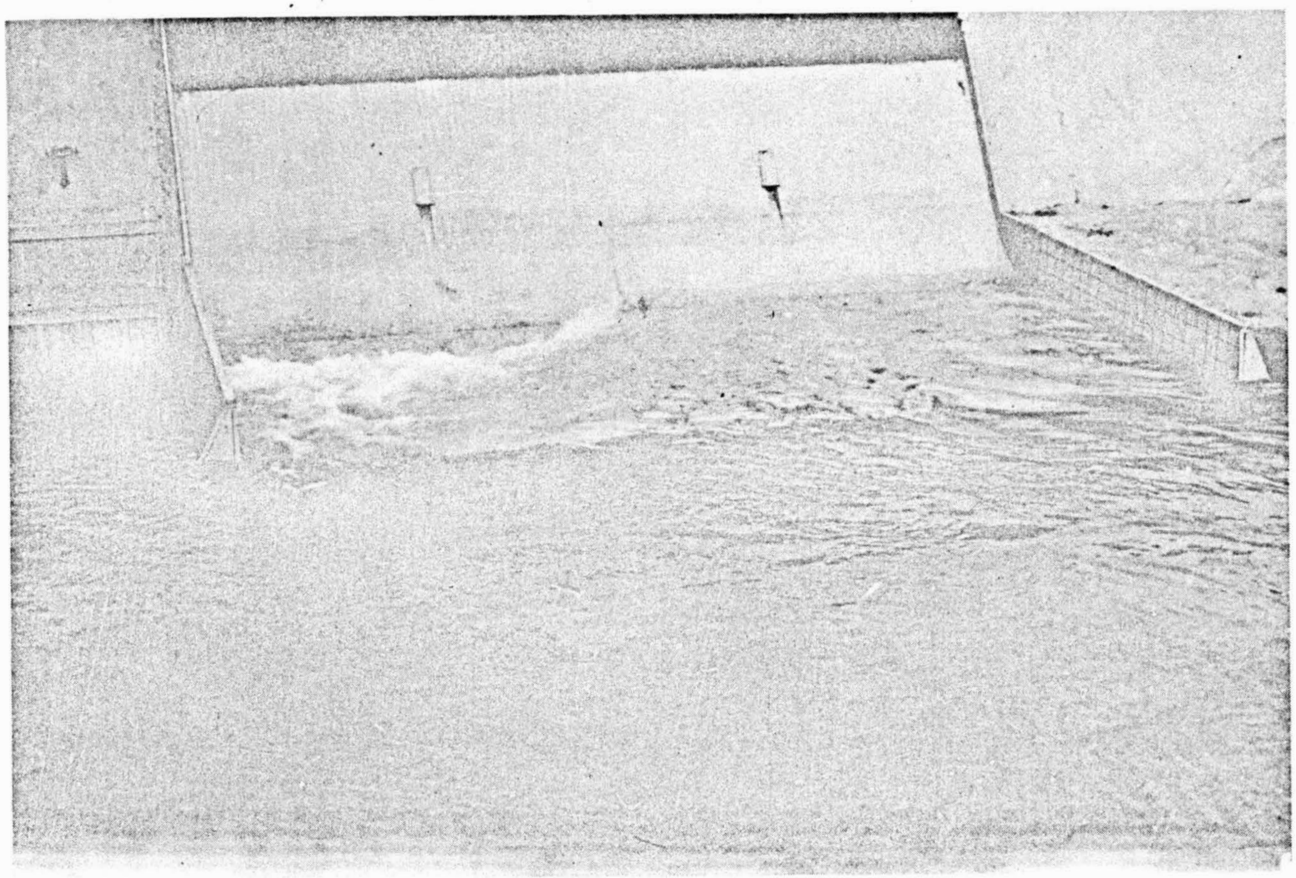


PLATE 24
Flow Patterns - Stilling Basin
View from Downstream
Gate Operation Schedule - Seven Right Gates Open 4 Ft
HW Elev 880.1, TW Elev 639.6, Q = 51,400 cfs
Spillway and Powerhouse Flow Only

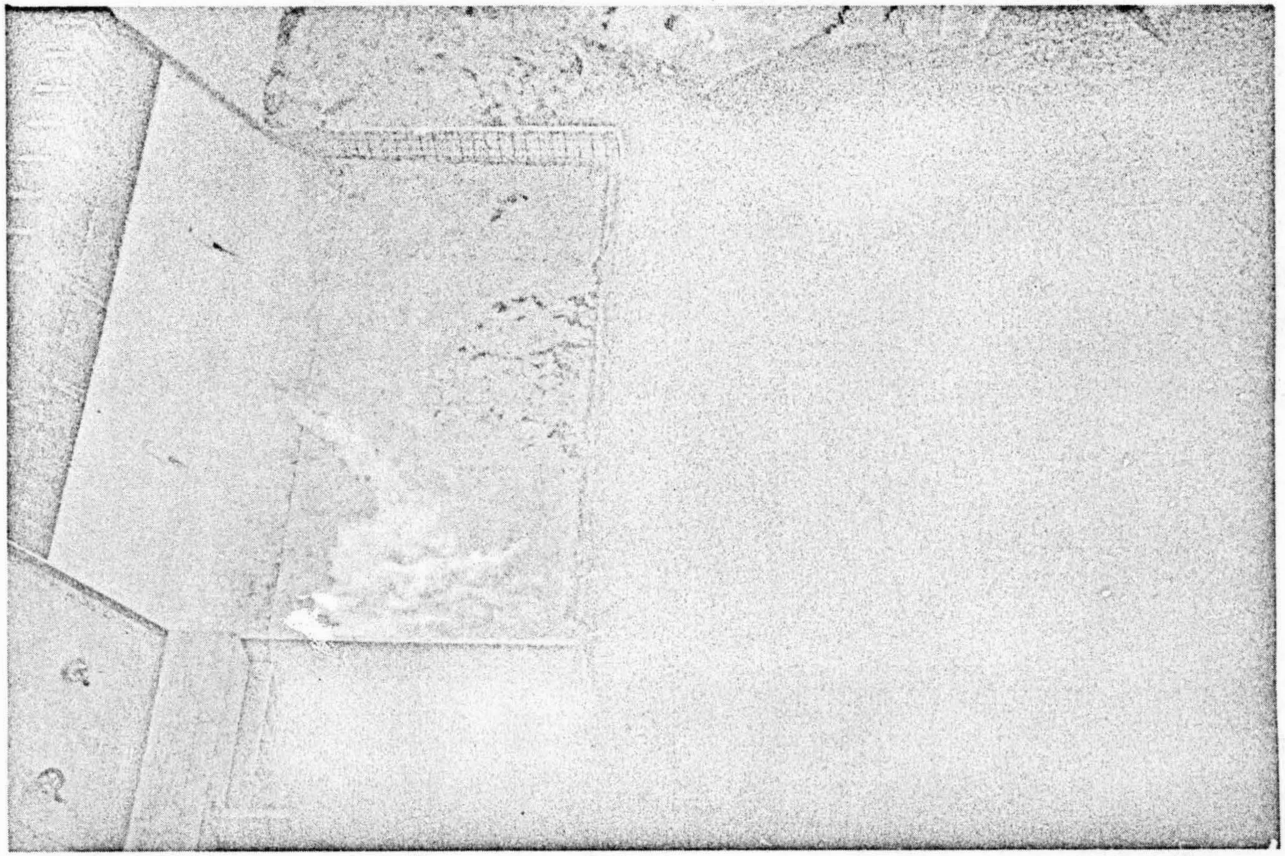


PLATE 25
Flow Patterns - Stilling Basin
Side View
Gate Operation Schedule - Seven Right Gates Open 4 Ft
HW Elev 880.1, TW Elev 639.6, Q = 51,400 cfs
Spillway and Powerhouse Flow Only

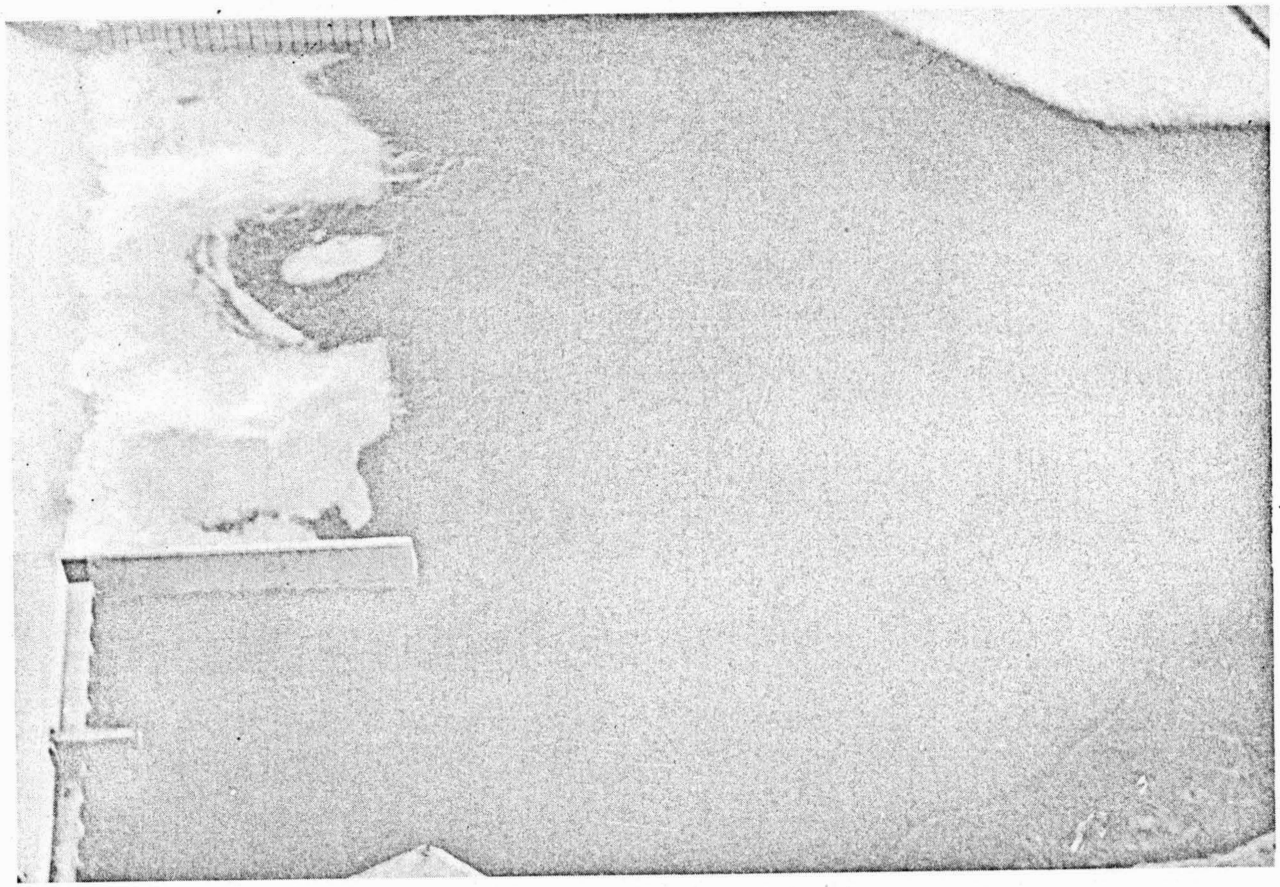


PLATE 26
Flow Patterns - Stilling Basin
Deposition of Bed Material
Pattern formed in the Downstream Channel caused by Eddy
HW Elev 880.5, TW Elev 644, Q = 69,500 cfs

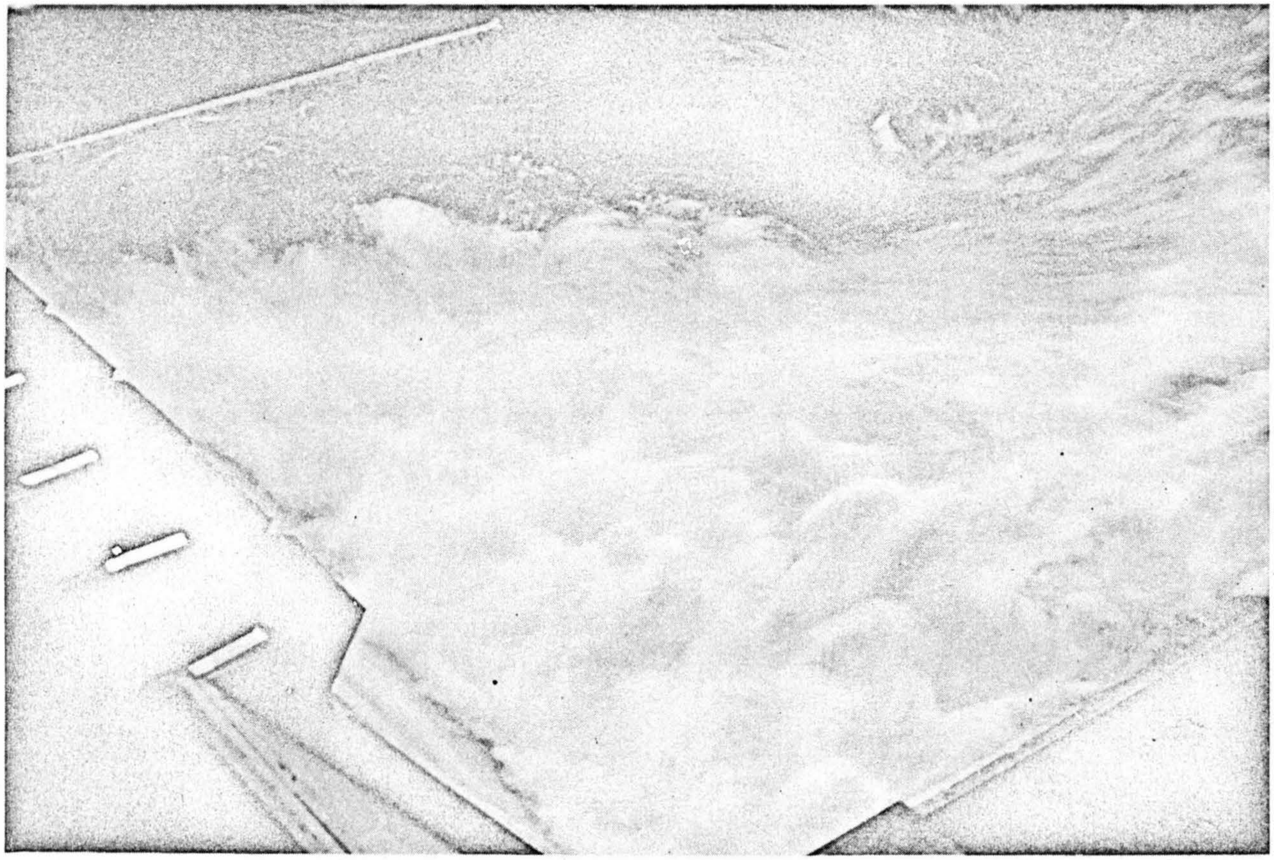


PLATE 27
Flow Patterns - Stilling Basin
Gate Operation Schedule - Two Left Gates Closed
HW Elev 888, TW Elev 685.5, Q = 390,000 cfs
Spillway Flow Only

PLATE 28
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
Side View
HW Elev 887.6, TW Elev 679.7, Q = 477,000 cfs

PLATE 29
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
Side View
HW Elev 887.6, TW Elev 679.7, Q = 477,000 cfs

PLATE 30
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
Oblique View from Left Side
HW Elev 887.6, TW Elev 679.7, Q = 477,000 cfs

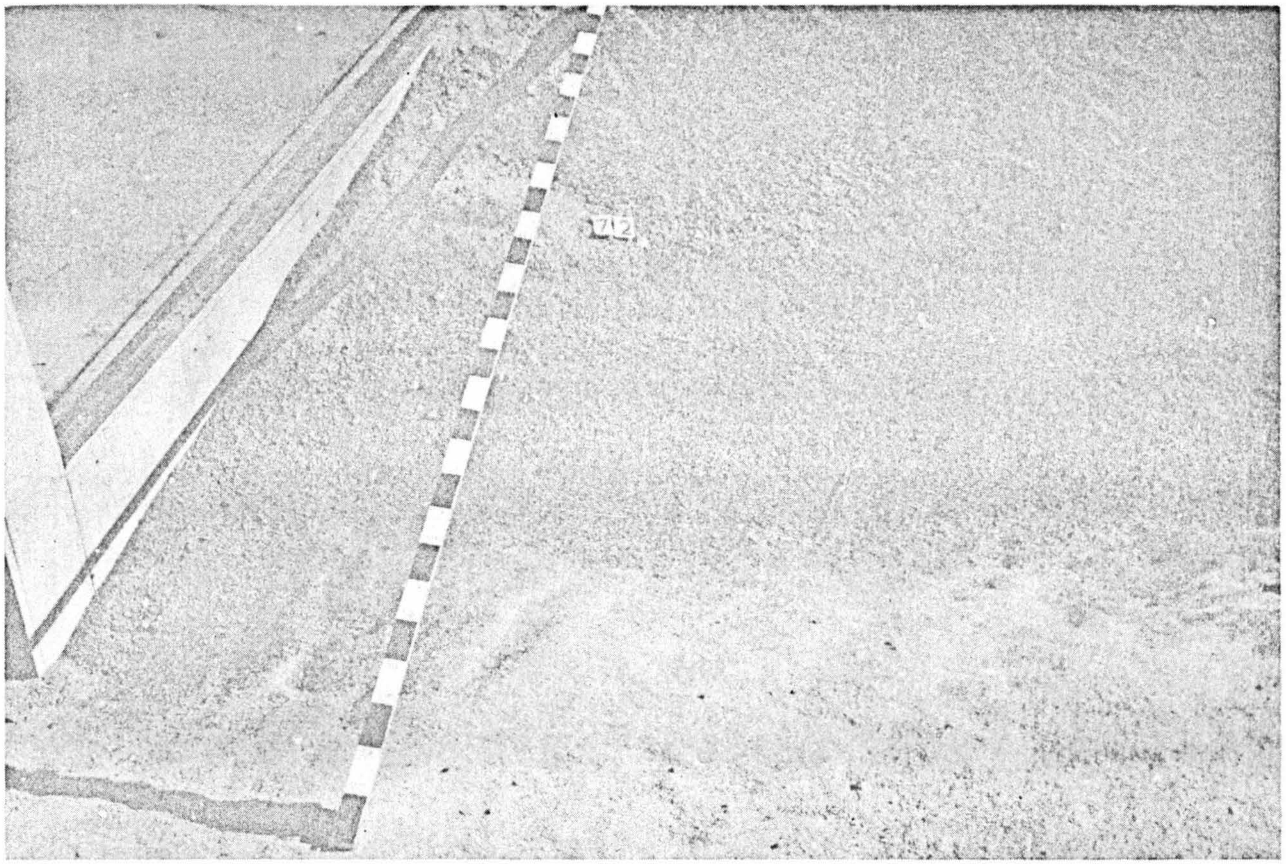


PLATE 31
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
View from Right Side
Gate Operation Schedule - Two Left Gates Closed
HW Elev 888, TW Elev 685.5, Q = 390,000 cfs,
Spillway Flow Only

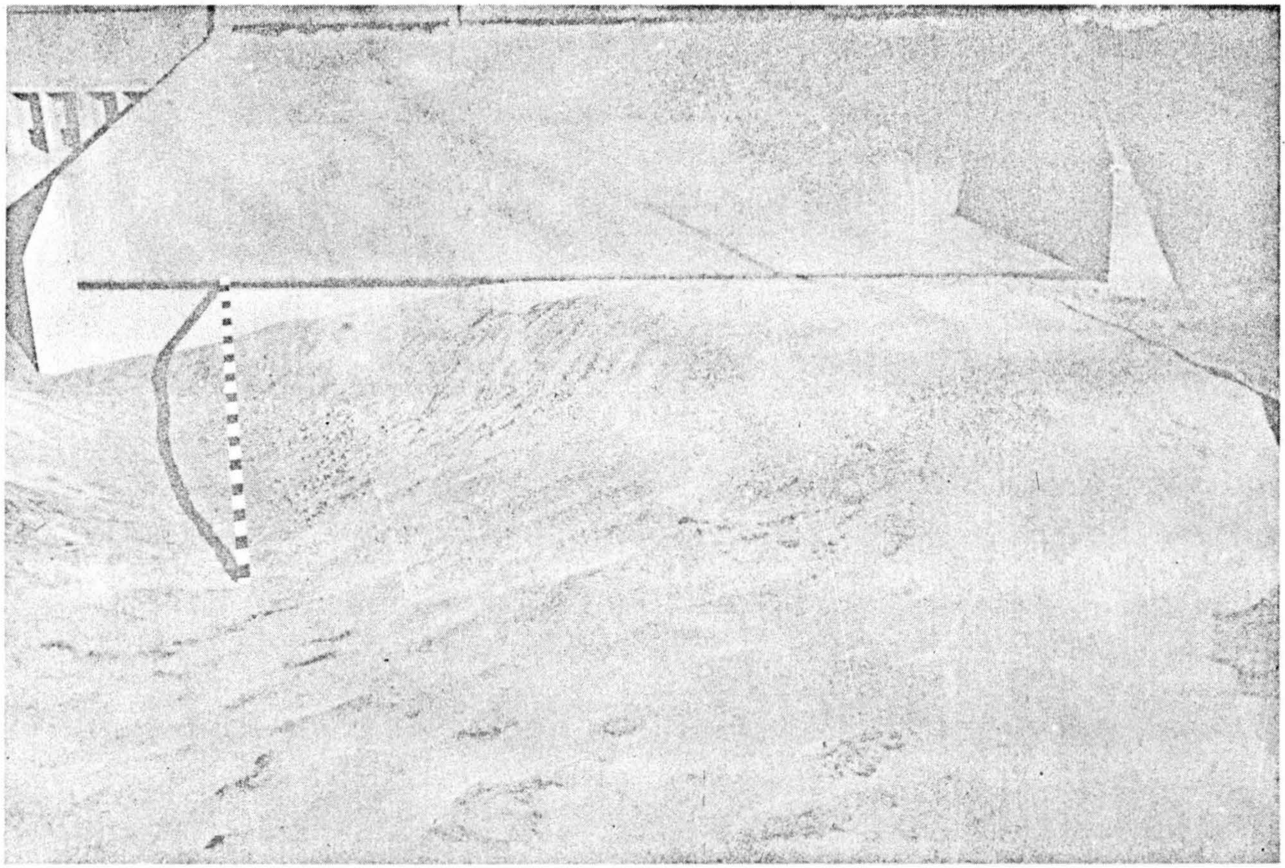


PLATE 32
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
View from Downstream
Gate Operation Schedule - Two Left Gates Closed
HW Elev 888, TW Elev 685.5, Q = 390,000 cfs,
Spillway Flow Only

PLATE 33
Erosion Tests - River Channel
Bed Movement Downstream from Stilling Basin
View from Left Side
Gate Operation Schedule - Two Left Gates Closed
HW Elev 888, TW Elev 685.5, Q = 390,000 cfs,
Spillway Flow Only

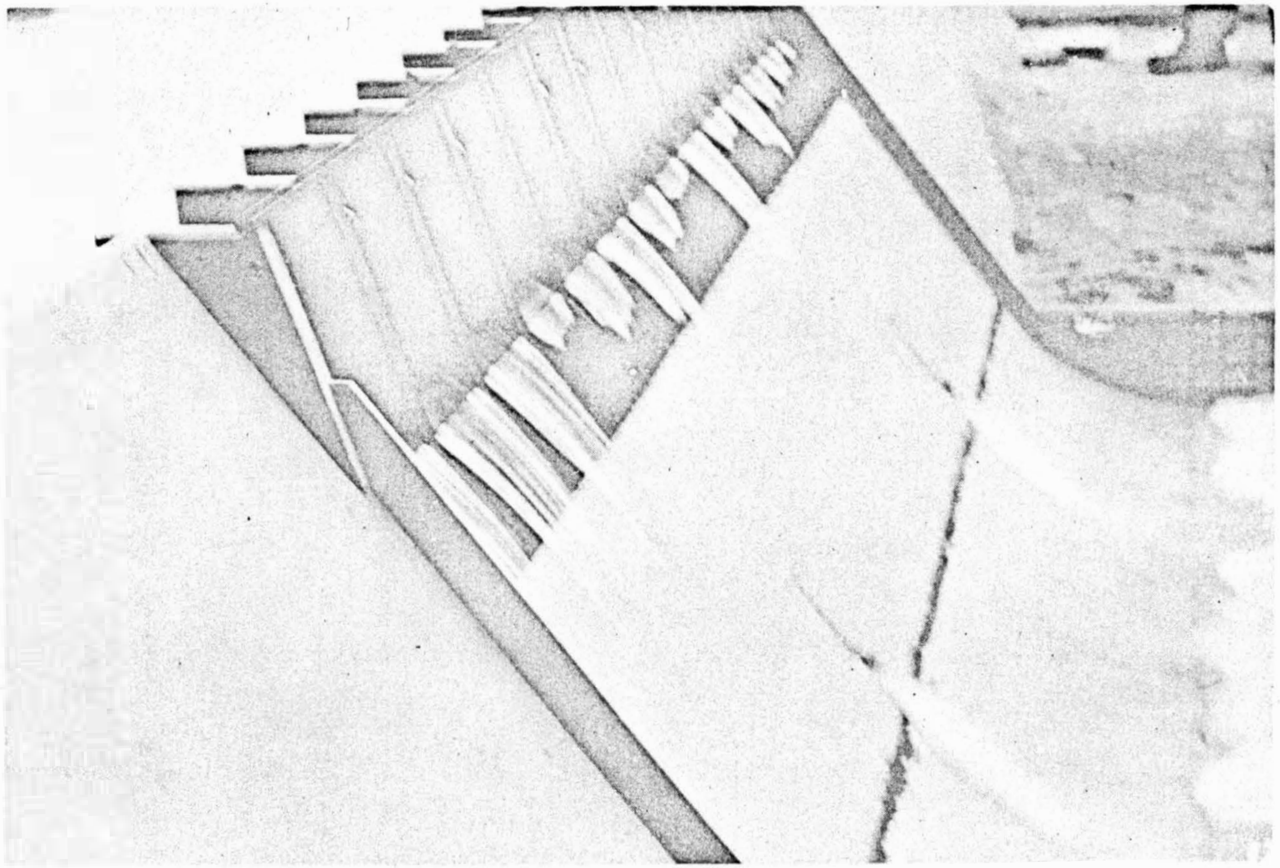


PLATE 34
Flow Patterns - Spillway Face
Profile of Flow over Air Step
HW Elev 878.6, TW 641.1, Q = 68,200 cfs, 2-ft Gate Opening