Government of India
Central Waterpower, Irrigation,
And Naviagation Commission
New Delhi, India

Report

on

HYDRAULIC MODEL TESTS

for

HIRAKUD DAM

Prepared by
Civil Engineering Section
COLORADO AGRICULTURAL EXPERIMENT STATION
Fort Collins, Colorado

for

INTERNATIONAL ENGINEERING COMPANY, Inc.
Denver, Colorado

January 1950

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FOREWORD

The studies described in this report were made during the period from April 1949 to December 1949 in the Hydraulics Laboratory of Colorado A & M College, Fort Collins, Colorado. Construction and testing of the Hirakud models was authorized in a contract between the Colorado Agricultural Research Foundation of Colorado A & M College through the Civil Engineering Section of the Experiment Station and the International Engineering Company, Inc. of Denver, Colorado.

The engineers of the International Engineering Company who were responsible for the design of the structure, particularly Messrs. B. M. Johnson, H. W. Birkeland, and W. A. Waldorf, studied the models in operation and discussed the results with laboratory staff members before and after the various revisions were made. Throughout the entire construction and testing of the models, consultations with and inspections by representatives of the company were maintained at regular intervals. These representatives were Messrs. S. H. Louie and W. A. English.

Several engineers from India who were representing the interests of the Indian Government inspected various aspects of the model studies and conferred with the laboratory staff regarding the tests. These engineers were Messrs. Kanwar Sain, S. C. Desai, Pritam Singh, and G. S. Subramanyan.

Since July 1949, Professor T. H. Evans has been Dean of Engineering and Chairman of the Engineering Division of the Experiment Station and since August 1949, Dr. Dean F. Peterson has been Chief of the Civil Engineering Section of the Experiment Station.

Laboratory staff engineers who contributed to the model studies were Messrs. A. R. Robinson, in charge of the design office and the construction of the models; D. Q. Matejka, in charge of the testing of the models; and King Yu and C. H. Zee, assistants to Mr. Matejka. During the summer, Mr. C. H. Lamb helped with the model studies and took a number of the photographs. Professor S. D. Resnick supervised the compilation of the material for the report. Other laboratory staff members were Messrs. James A. Decker, construction foreman; and Lyle A. Wiggen, machine-shop foreman.

Part-time staff member of the laboratory was Mr. A. J. Peterka, technical consultant, who was assisted by Messrs. W. E. Wagner and Y. Fong. Mr. Peterka assumed considerable responsibility in establishing the design of the original models and in initiating revisions as well as in organizing and writing the report.

The entire program was under the direct supervision of Dr. Maurice L. Albertson.

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Chapter I

INTRODUCTION

General

Hirakud Dam will be constructed on the Mahanadi near Sambalpur in the State of Orissa, by the Central Government of India. This site, with an elevation of about 500 feet above sea level, intercepts a watershed of 32,200 square miles, and the reservoir created will have a storage capacity of 6,000,000 acre-feet at reservoir Elev. 625. The project will produce hydro-electric power, provide storage of irrigation water, have locks for the passage of shipping, and give limited flood control on the lower Mahanadi. A region encompassed by approximately a radius of 200 miles will receive direct benefits from the project. General relative locations are shown in Dwgs. 1 and 2.

Need for Studies

The Central Waterpower, Irrigation and Navigation Commission, the agency of the Government of India responsible for the Hirakud Project, procured the services of the International Engineering Company, Inc. (Denver, Colorado office) to design the dams and appurtenant works. Dr. John L. Savage was retained as consulting engineer for the project.

Due to the uniqueness and over-all magnitude of this project, hydraulic model studies were deemed essential for proper design by the Central Waterpower, Irrigation and Navigation Commission, the designers, and the consulting engineer. Accordingly, the Colorado Agricultural Research Foundation of Colorado Agricultural and Mechanical College was engaged by contract with the designer in April, 1949 to perform the model studies and submit a report on its findings. During the ensuing period this work has been conducted in the Hydraulics Laboratory of Colorado A & M College at Fort Collins, Colorado.

Scope of Investigations

The primary objective of hydraulic model studies is to aid the designer in developing and checking established concepts of the proposed hydraulic features, and thereby provide assurance that such structures will operate as intended. An over-all model of the project would have been desirable to integrate the performance of the component parts of the project, and a model of the proposed navigation locks would have been helpful in obtaining a satisfactory and economical hydraulic filling and emptying system. However, limitations in time and funds restricted the investigation to studies of a 4-1/2-bay, sectional model of the spillway and sluices. The auxiliary spillway apron and the service spillway apron were investigated and developed from the tests on the sectional model. Two types of sluices, open channel and pressurized, were investigated and finally a separate model of one sluice inlet was constructed and tested to develop a more satisfactory inlet shape.

It was decided that this report should contain not only the usual interpretation of data and recommendations, but also a detailed description of the entire process of making and interpreting the model studies. Thus, this report has been expanded to include the reasoning used in determining the model scales, the explanation of the design and construction of the models, and the testing procedures.

Prototype Structure

Hirakud Dam will be constructed across the Mahanadi at a point where the river consists of two primary channels (see Dwg. 3). Spillway flows will be discharged into the left channel and the power dam and navigation locks will be located in the right channel. There will be no intermingling of flow in the two channels. The entire structure, spanning both channels, will be about 16,000 ft long. The structure will consist of a concrete spillway dam 3,270 ft long flanked by earth dams totaling 9730 ft in length and the concrete power dam and navigation locks which total 2800 ft in length.

Spillway Dam - The general plan of the spillway dam and its location relative to the river and the topography of the site is shown in Dwg. 4. It is a straight-alignment, concrete gravity structure with maximum height approximately 240 ft. The downstream face of the dam has a slope of 1 on 0.7. Flood discharges will be passed through 84 sluices and 86 crest spillway bays. As may be seen in Dwgs. 5 and 6, the barrel of each sluice is 20.33 ft high by 12 ft wide and the flow is controlled by a vertical wheeled gate. Each bay of the crest spillway is 21 ft wide and in the original design was controlled by a radial-type crest gate having an effective height of 12 ft. In the final design, however, the crest gate has been changed to a vertical type.

The 84 sluices (70 operating) have a discharge capacity of 1,070,000 cfs and the 86 bays of the overfall spillway 250,000 cfs for the normal reservoir Elev. 625, making the combined capacity 1,320,000 cfs. Thus each sluice will discharge 15,300 cfs and each spillway bay 2,900 cfs. To provide for the handling of large floods the structure is designed to handle discharges up to the maximum water surface Elev. 630. The designers planned that for this head each sluice would discharge 15,500 cfs and each spillway bay 5,200 cfs making a combined total of 1,535,000 cfs.

The spillway is divided into two separate sections, a service section which will be operated until its capacity is exceeded, and an auxiliary section which will be used to pass the larger floods. In the service section, which has 17 crest spillway bays and 16 sluices, the energy developed by the fall of 160 ft is 5,520,000 HP total or 13,100 HP per ft of width. This energy is to be dissipated in a stilling basin having a horizontal apron and a dentated end sill as shown in Dwgs. 5 and 7.

Below the auxiliary spillway section, which includes 69 crest gates and 68 sluices, there is no stilling basin to dissipate the energy but rather a curved trajectory bucket having a radius of 50 ft which projects the water downstream as far from the structure as possible (see Dwg. 5). The downstream river channel is composed of rock either exposed or thinly covered with loose material.

A set of area-capacity and discharge rating curves, for the reservoir and appurtenant structures, is given in Dwg. 8.

Power Plant - A power plant, housing six hydro-turbine generators of 37,500 kw capacity each, will be constructed contiguous with the concrete, gravity type power dam (see Dwg. 3). Penstocks, controlled by fixed-wheel gates within the dam, extend through the dam to serve the turbines. The flow leaving the tailrace is turned to the left by training walls.

Navigation Locks - In order to provide for navigation on the Mahanadi past Hirakud Dam, navigation locks will be constructed near the right end of the power dam. The double lift locks will raise or lower shipping a maximum distance of 116 ft. Ships having a 9-ft draft can pass through the locks which are 60 ft in width and 360 ft in length. Approximately 500 ft downstream from the locks, the navigation channel is turned 45° to the left (Dwg. 3).

Model Studies

Preliminary to constructing models of Hirakud spillway, the hydraulic features of the proposed design were studied to determine where difficulties might be encountered in obtaining satisfactory performance. As a result of this analysis it was found that the problems were of two general types--those involving the flow passages and those pertaining to the structure as a whole. Those of the first type; flow over the crest spillway, flow through the sluices, and performance of the apron and trajectory bucket were considered of primary importance. Those of the second type; flow in the river both upstream and downstream from the dam, effect of the service and auxiliary spillways on flow in the river channel, necessary height and length of the training wall between the spillways, and wave effects in the lower channel also needed to be studied.

To test and develop the structure in an ideal manner, it would have been desirable to study problems of the first type using separate models for each individual part of the structure in so far as possible. The developed designs of each feature could then be assembled into a complete model of the entire project where the problems of the second type could be studied.

In the case of the Hirakud project, however, it was not practical to construct one model to study both types of problems. The more detailed problems of the first type require larger models than do those of the second type. Because of the great length of the dam relative to the height, a model large enough to study in detail the individual flow passage would be of great size, and much larger than necessary to study the river flow conditions. Data would be difficult to obtain inside a model of this type and if changes were made in the design of the sluices or spillway bays, extensive changes throughout more than 80 additional sluices and bays would be necessary. It was decided, therefore, that a relatively large sectional model of the spillway dam should be constructed to study the performance of the individual flow passages and the flow pattern in the stilling basin immediately downstream from the dam. The model scale was selected after taking into account: (1) the type of data which was necessary to evaluate the designs, (2) the absolute dimensions of the closed conduit portions of the model, (3) the absolute depth of water on the spillway crest, (4) the degree of accuracy necessary in the model, and (5) the laboratory facilities such as the capacity of the pumping system and the floor space and head room available. A model scale of 1:40 was found to fulfill all the requirements. To reduce the sidewall effects in the test flume to a minimum, it was necessary to construct a section of spillway 4-1/2-bays wide. By proper design it was possible in the same flume to study, with only minor changes in the model, both the service spillway and the auxiliary spillway.

Although it would have been desirable to build and test, in addition, a smaller complete model to study problems of the second type, limitations in time and funds prevented such a study for the present. Construction and testing of a model of the navigation locks was prevented for the same reasons.

The first tests on the sectional 1:40 scale model to determine the pressure distribution in the sluices indicated that extensive development studies would be necessary. To simplify and reduce the cost of developing a sluice design it was necessary to construct and test an auxiliary model of one sluice inlet. This model, also built to a scale of 1:40, was arranged to make it possible to obtain basic data that could not be conveniently obtained from the sectional model.

Chapter II

THE SERVICE SPILLWAY MODEL

A. MODEL CONSTRUCTION AND TESTING

Before describing the model construction and testing, it is necessary that the problems to be considered in the model tests be thoroughly understood. The anticipated problems affect the design of the model as much as any other factor. For example, if it is suspected that a portion of the structure may not perform satisfactorily, the model should be made flexible at this point. Therefore, in this section the problems to be considered will be presented first, followed by a description of the design and construction of the original model, and the equipment and procedure for testing. In other sections the test results and the recommended revisions will be discussed.

Problems to Be Considered

In order to systematically present the problems which must be considered in the study of the service spillway model, they will be discussed in order starting with the entrance and exit conditions, followed by the performance of the spillway, the sluice, and the stilling basin.

Entrance and Exit - In most model studies, it is very important to reproduce the approach conditions upstream from the dam. However, as may be seen in the general layout of the original design (Dwg. 3), the topography upstream from the dam is sufficiently far from the service spillway to have very little effect on the flow approaching the spillway. Thus, in the sectional model it is sufficient to provide an approach channel scaled approximately to the prototype depth. It also is necessary to provide a uniformly distributed water supply at the upstream end of the approach channel.

In the downstream portion of the sectional model, it should be possible to reproduce accurately the range of tailwater elevations indicated by the tailwater rating curve in Dwg. 8. With a given number of gates in operation in the sectional model, the same or any greater number of gates could be considered to be open in the prototype. Therefore, for a single arrangement of gates in the model it is necessary to test the model over a wide range of tailwater elevations.

Overfall Spillway - The items to be considered in connection with spillway performance are the flow patterns over the crest and adjacent to the piers, on the face of the spillway, and in the vicinity of the sluice outlets. Because prototype operation of the spillway at times will involve various combinations of open and closed gates, the effect of one or more closed gates adjacent to an open gate or gates should be determined. When a group of spillway bays are operating together, the width of reservoir contributing to the flow through one of the central bays isonly the width between the extended centerlines of the piers. On the other hand when only a single bay is operating, the area contributing is much wider, thereby causing a greater degree of contraction of the flow as it enters the bay. The effect of the greater contraction on the discharge through a single bay should be investigated.

Although sufficient information is now generally available to enable designers to calculate an efficient spillway crest, it is advisable to check the pressures on the crest of the model. Excessively high pressures indicate an inefficient crest with a lower than necessary discharge coefficient. Very low pressures, on the other hand, indicate that cavitation could occur in the prototype.

Because of the various phenomena involved in the flow of water over the crest and down the face of the spillway, the problems of making the model completely similar dynamically to the prototype is very complex. Although the gravitational forces reflected in the Froude number dominate, the forces of viscosity in the Reynolds number also play a part in the flow problem.

Normally, as a fluid moves along a boundary, a boundary layer is formed. The thickness of this layer and the velocity distribution within it depend on the viscous forces within the flow and the roughness of the surface. Hence, as the water enters a spillway, flowing over the crest and past the piers, the boundary layer begins its development. In this particular zone, the streamlines converge rapidly due to the acceleration caused by the gravitational forces and tend to confine the boundary layer so that the viscous forces become relatively less important. Therefore, in the region of the crest the gravitational forces represented by the Froude number are an adequate criterion for similarity.

As the water progresses down the face of the spillway, the streamlines converge less rapidly and the boundary layer is therefore able to develop in a more normal fashion. As previously described, this development depends upon the Reynolds number of the flow and the relative roughness of the surface. Which of these two factors predominates in its influence depends upon the relative magnitude of the forces created by each. If viscous forces predominate, the roughness is covered by the laminar sub-layer and is therefore ineffective. In this case Reynolds number must be a constant from model to prototype in order to have absolute dynamic similarity. Unfortunately, however, if water is used in the model, it is impossible to have both the Froude number and the Reynolds number a constant at the same time. Thus, it is impossible to completely reproduce prototype flow in the model.

If roughness predominates, so that the laminar sublayer is destroyed, the turbulence and general development of the boundary layer become independent of the Reynolds number, and are a function only of the relative roughness. In this case, then, it is necessary to keep both the Froude number and the relative roughness a constant from model to prototype -- a condition which is possible at least theoretically.

For these reasons, care should be taken in choosing the scale of the model and in the design of its various features to make the gravitational forces predominant over the viscous forces and to have the roughness as near as possible geometrically similar to the prototype roughness. To be certain that gravity forces predominate, the model should be constructed sufficiently large that the forces involved in the development of the boundary layer and the surface tension forces are relatively insignificant. A rule of thumb sometimes used for establishing the scale of a model is to have the head on the crest of the overfall spillway greater than 2 inches, the width of the bays greater than 3 inches, and sluices 4 inches in the least dimension.

Finally, the discharge capacity of the spillway should be determined for various headwater elevations. Rating curves for a single gate operating and also for all gates operating should be obtained both to aid in properly operating the completed prototype structure and also to assure the designer that the capacity of the structure meets the necessary requirement.

Since the powerhouse is far removed from the spillway (see Dwg. 3), the conditions resulting from excessive spray near and on the powerhouse and switchyard were of no concern. Therefore, studies to measure the amount of spray were not initiated.

Sluices - To evaluate the performance of the sluices, it is necessary to study and evaluate the flow pattern through the sluices, taking special note of flow at the entrance and throughout the barrel of the sluice. At the outlet, the characteristics of the flow pattern should be determined with the sluice operating alone and also with flow from the spillway crest as it impinges upon the water discharging from the sluice. A glass sidewall or panel for observing this flow pattern visually is extremely advantageous and should be provided. Because the flow patterns can be disturbed appreciably by minor irregularities in the model, it is of primary importance that the sluice and the gate which controls the flow be reproduced very accurately in the model.

In order to provide a sufficient number of sluices through the dam to pass the large discharges which occur during a flood, it was necessary for the designers to place the sluices so close together that the design of the sluice inlet became a major problem. As a result of this situation, careful design was required in order to obtain satisfactory pressures in the sluice inlet. It is necessary, therefore, to provide means in the model for accurately measuring the pressure distribution at all critical points in the sluice.

To be certain that the discharge through the sluices is sufficient to meet the requirements and also to provide information for operation of the completed prototype structure, the discharge capacity of the sluices should be determined for various headwater elevations.

Stilling Basin - The energy per foot of width entering the stilling basin of the service spillway is unusually large. An efficient stilling basin is therefore a necessity. Tests on a number of designs are usually necessary to develop and prove the effectiveness of a stilling basin. Therefore, the model basin should be sufficiently flexible that modifications can be made without difficulty.

Within the stilling basin, the general action should be observed and analyzed. Waves and surges created in the downstream channel as well as the erosion should be observed and measured. Thus, a movable bed to measure the depth and extent of the erosion should be provided and staff and point gages should be available to measure the wave heights.

Obviously, with a sectional model it is possible to reproduce the transverse flow pattern only to a very limited extent. Investigations should be made of unsymmetrical gate operations which cause eddies and the deposition of debris in the bucket of the auxiliary spillway. Action of this type might erode the prototype bucket to a dangerous degree.

Pressure on the dentated end sill should be checked to be certain that dangerously low pressures do not occur on the sill corners. The model pressures are also useful to the designers in providing for sill stability in the prototype. Means should thus be provided in the model for measuring the pressure distribution on the sill.

Design and Construction of Original Model

After consideration of the foregoing problems, the original model of the service spillway was designed and constructed. The discussion of the model design and construction has been divided into the following headings: the general layout of the model, the head box and glass-walled test flume, the river bed, the overfall spillway, the sluices, and the stilling basin.

General Layout - The 4-1/2-bay sectional model of the spillway was constructed in a test flume with a glass sidewall in the upstream end through which the performance of the spillway, sluices, stilling basin, and a portion of the downstream channel could be viewed (Plate 1 and Fig. 1). At the upstream end of the flume, a head box equipped with a stilling baffle prepared the flow from the laboratory supply line to pass over or through the structure.

At the lower end of the test flume a tail gate was constructed to provide the necessary control of the tailwater elevation. The test flume was made sufficiently long to develop the flow and scour patterns which would exist downstream from the stilling basin when the maximum discharge was passing through the model (Plate 2). The height of the flume was governed by the maximum height of the tailwater plus surges and splash, and the lowest expected depth of scour. Sufficient depth for scour was provided so that the wood bottom of the test flume would not be exposed during a test.

The height of the head box above the top of the spillway crest was governed by the expected head loss through the stilling baffle plus some freeboard. The elevation of the top of the head box was approximately elevation 659, making the head box 6.2 feet high. The equivalent of 40 feet of bed material below the apron at Elev. 452, and 60 ft below the top of the original end sill was provided. The top of the flume sidewalls was approximately Elev. 575, making the flume walls 4.75 ft high in the model.

Head Box and Glass-Walled Flume - The head box, 8 ft by 7 ft and 6.16 ft deep, was made wider than the test flume to provide low-velocity tranquil flow at the entrance to the flume. At the upstream end of the head box a rock baffle 6 inches thick (Fig. 1) was placed across the entire width of the head box to dissipate the energy of the incoming flow of water from the 8- and 14-in. inlet pipes of the circulation system. A transition (Fig. 1 and Plates 3 and 4) was used to conduct the flow from the head box to the narrower test flume. The transition curve was designed to provide uniform flow conditions in the spillway approach.

In order to reduce the cost of connecting the model to the water supply, a low-priced, low-head irrigation valve was used instead of the more expensive standard valve which is built to withstand pressures of 150 psi. Because this valve was originally made for use with 14-in. concrete pipe, an adapter was built in the laboratory so that the valve could be used with 14-in. steel pipe having 1/8-in. walls. To connect the pipe to the head box, a flange was welded to the end of the pipe and bolted to the wall of the head box. A standard 9-in., high-head valve was used in the auxiliary 8-in. supply line.

As explained previously, it was necessary to construct the head box 6.16 ft high. This height of water produced a load on the floor of 385 psf, which made it necessary to build the floor of the head box on a firm footing and to reinforce the sides with special 2-in. by 6-in. whalers and 1/2-in. steel tie bars through the inside of the box (see Plate 5). To prevent movement of the head box sidewalls and the resulting failure of the water seals due to hydrostatic pressure, two of the tie rods were placed as close to the spillway as possible without influencing the flow pattern of the water over and through the model.

To simplify and speed construction and yet have watertight fabrication, the framework of the head box and test flume, made largely of 2-in. by 4-in. joists and studs, was lined with water-proof plywood 1/2-in. thick (see Plate 6). Previous experience had shown that the use of plywood provided the most economical method of obtaining a water-tight box. This was particularly true at the time of the Hirakud construction, when sheetmetal -- an alternate lining material -- was very expensive and difficult to obtain. Since the standard size for plywood is 4 ft by 8 ft, the box was designed with the studs and joists placed so as to reduce waste to a minimum.

A special method was developed to seal the joints. As shown in Plate 6, sheets of uncured rubber were cut into strips, then doubled and cemented securely into place in each joint. Powdered talc and strips of the coated cloth in which the rubber had been wrapped were placed in the fold to prevent the rubber-to-rubber faces from sticking together. Approximately 1/2 in. of the loose edge of the rubber was left outside the joint and was later cemented to the adjacent wall or floor. With proper care in making this joint, it was possible to provide an absolutely tight and water-proof seal which had no tendency to leak even under high heads. The natural shrinking and swelling of the wood was accommodated by the fold or pleat in the rubber. After construction was completed, the entire inside of the box was given two coats of oil paint to protect it from the water as much as possible.

The construction of the test flume was performed in much the same manner as the head box, tie rods being installed at 6 ft intervals to eliminate the possibility of movement due to hydrostatic pressure. The flume was made 30 ft long to allow flow and scour patterns to be completely developed. In order to view the action of the hydraulic jump, the flow patterns, and the extent of bed movement occuring under specific flow conditions, a glass section 16 ft long was installed in the flume sidewall extending 1 ft upstream from the axis of the dam and 15 ft downstream. The glass-walled section consisted of three sheets of 4 ft by 6 ft tempered plate glass 1/2 in. thick mounted in a frame of 1-1/2-in. by 3-1/2 in. structural steel channel. One sheet was mounted vertically and the other two horizontally as shown in Plate 1 and Fig. 1. The upstream and higher portion of the glass wall which was subjected to head-box pressures was reinforced with both 1-1/2 in. by 3-in. channel and 1-1/2-in. by 1-1/2-in. angle to prevent excessive deflections.

At the downstream end, on the floor of the test flume, a bulkhead was constructed to Elev. 492, which was the elevation of the prototype river bed at this point. The bulkhead was used to prevent the river bed material from spilling out of the end of the flume. It also formed the upstream side of the bed material trap. The trap was 2-1/2 ft long in the direction of flow, and was intended to trap eroded material which otherwise would have been carried into the laboratory reservoir. A tailgate was built downstream from the trap to supply a flexible control of the tailwater. It was regulated by two cables connected to a winch mechanism made of 1-1/2-in. pipe which operated in wooden bearing blocks made of 2-in. lumber. The resistance in the bearings could be adjusted by bolts with wing nuts which held the top half of the bearing in place. Crank arms were fastened to each end of the pipe to permit easy operation. Beyond the tailgate a chute was constructed to guide the water into the return channel of the circulation system.

River Bed - The river bed in the Hirakud model was molded to the shape of the river bottom downstream from the structure as shown in Plate 21. Since the sectional model represents a slice out of the center of the service spillway, no river banks were modeled. The erodible material used in the model was comparable to 1.00-ft diamater material of the prototype, as determined by the fall-velocity method which is explained later in detail. The erodible portion of the river bed comprised an area of 2.81 by 15 ft in the model, or an area of 112.5 by 600 ft in the prototype. The maximum depth to which the model river bed could be eroded was 60 ft, prototype.

Overfall Spillway - For Hirakud Dam, it is probable that over the smooth portion of the spillway crest and face, the effect of the Reynolds number is minor. As the flow progresses down the spillway face, the boundary layer becomes more fully developed, thereby decreasing the thickness of the laminar sub-layer and making the Reynolds number even less important.

Because of the impossibility at this time of knowing the prototype surface roughness and because of the limited selection of materials available for model construction, it is difficult to reproduce in the model the roughness of the prototype surface. However, in view of the fact that the spillway is steep and the length of the spillway face is not appreciably longer than the vertical height of the spillway, the type of flow in the model is a reasonably accurate representation of the flow which will occur in the prototype. Thus, it is reasonable to assume that, for the Hirakud spillway, the Reynolds number is unimportant and that the flow in the model will be similar to that in the prototype in so far as mean velocity, velocity distribution, and turbulence are concerned.

The model of the Hirakud spillway was designed to simplify the construction as much as possible and to divide the model into units that could be accurately constructed. Construction joints were provided in many places so that the revision of one portion would not unduly disturb other portions which required no modifications. Although no modifications were made on the crest and face of the spillway, provisions were made in the model to make changes, had they been necessary.

The face of the spillway (see Plate 7) was made of 1/2-in. plywood fastened to a 1-1/2-in. by 1-1/2-in. steel angle framework designed to hold the sheet metal sluiceways and support the concrete crest section. The outer face of the plywood was covered with 18-gage sheet metal to insure a smooth spillway surface and to facilitate attaching the sheet metal sluiceways to the downstream face by soldering. The steel framework extended from the floor of the flume, Elev. 412, to Elev. 574.25, to permit construction of the spillway crest on a shelf placed on top of the framework, as shown in Plate 8.

It was decided that concrete should be used to construct the spillway crest in view of the fact that wood would swell and warp out of shape after it had been repeatedly wet and then dried. To use concrete it was first necessary to provide sheet metal templets cut to the shape of the crest profile and mounted on the framework shelf. The concrete was then finished smoothly and accurately to the profile of the templets. To make the templets, 18-gage sheet metal was painted with a special lay-out fluid which would permit accurately scribing the shape of the crest on the metal, using drafting instruments and procedures. Because of the width of the crest, six templets were required. After marking each piece of metal, the templets were cut just outside the scribed line with a metal-cutting band saw and then carefully finished to the center of the line by hand filing.

In order to hold the templets in place with proper alignment, they were threaded onto three long bolts with pipe sleeves over the bolts acting as spacers for the templets. This unit was then fastened on top of the shelf. A strip of metal shaped to the crest profile and containing the piezometers (see Plate 9) was fastened to one of the templets so that pressures on the crest could be measured. The templet to which the piezometer assembly was fastened was undercut sufficiently so that after the piezometer assembly had been fastened to it, the profile was correct. The piezometer tubes, made of 1/8-in. O.D. brass tubing with a 1/16-in. diameter hole were soldered to the under side of the cap and to the side of the templet itself, as shown in Plate 10. This gave sufficient rigidity to the piezometer assembly and ensured that no changes in the profile would occur either while concrete was being placed or later during testing. The piezometer tubes were carried down through the crest and out the side of the test flume to a manometer board.

Before the crest was poured, 8-1/2-in. lengths of 1/2-in. pipe were placed vertically in the crest form to provide a passageway for the rods used to open and close the sluice gates (see Plates 10 and 11). The sluice gate operating rods were extended upward through the crest because it was impossible to include a sluice gate gallery with a practical type of gate operating mechanism inside the 1:40-scale sectional model. By use of the above-described method, it was possible to operate the sluice gates from above the spillway model, this being possible because the operating rods emerged from the tops of spillway piers and were not in the line of flow.

The concrete crest was poured in three separate operations. The base coat was first poured to within about 1/2-in. of the top of the templets and allowed to set for 24 hours. Next, the scratch coat was applied, being poured to within about 1/8-in. of the top of the templets -- the workman making certain that a very rough, or scratched texture remained so as to provide a good bond surface for the final or finish coat to be applied after the second layer of concrete had set. Both the base and the scratch coats consisted of 1 part Portland cement and 3 parts sand, and were permitted to set completely so that nearly all shrinkage had taken place before the final surface coat was applied. The surface coat was made of 2-1/2 parts of Portland cement to 1 part of molding plaster. Enough water was added to make the mortar easy to work. The molding plaster gave the surface coat quick-setting properties so that it could be finished soon after placing. The surface coat was placed slightly above grade and immediately after the initial set, a matter of minutes, it was screeded to the shape of the templets and allowed to attain its final set before finishing. Final finishing, which was delayed for three or four hours, was done by sprinkling dry Portland cement on the surface and troweling until a very smooth surface was obtained (see Plates 7 and 11). This procedure required considerable skill and experience on the part of the operator to prevent trowel marks but was found to make an excellent smooth crest when done correctly. The piezometer holes were temporarily plugged with nails or wax while the crest was being made to avoid filling the openings with concrete.

Before the base coat of concrete for the crest had set, a sufficient number of 3/8-in. bolts were embedded in the concrete to provide each pier with an anchor bolt at both its upstream and downstream ends (see Plates 8 and 11). The piers were made of Honduras mahogany wood and given a waterproofing treatment to reduce as much as possible any swelling or warping. The piers were cut to the proper width and to the shape of the crest using a jointer and band saw, but the pier noses were shaped using hand tools. Sheet metal templets were used as gages to check the pier nose shape.

The face of each radial crest gate was made of 20-gage sheet metal, formed to an arc of 5.406-in. radius in a sheet metal roller. Prior to this the face plates were cut and squared so as to produce the required gate dimensions when formed. The gate arms were made of 20-gage sheet metal, extreme care being taken to ensure proper alignment of the gate and the correct radius of curvature of the end to be connected to the gate face. The gate arms were all clamped together when drilling holes for the gate pins, and forming the proper radius of curvature of the gate arm. This was done to be certain that all gate arms, and finally all gates, would be of uniform dimensions.

Small sheet metal tabs attached to the lower portion of the gate face were used for fastening thecables of the gate-opening device. This consisted of a winch arrangement similar to the one used on the tailgate. All gate cables were fastened to the winch and all gates were opened or closed simultaneously. The gates were assembled by a process of first bolting the gate face and the arms together with clip angles, and then soldering the joints carefully to prevent heat warping.

In order to prevent leakage past the model in the test flume, it was necessary to devise a method of sealing the model to the plywood side wall, as well as to the glass wall of the flume. Thin strips of sponge-rubber, ordinarily used for weather stripping were fastened around the edge of the model between the model and the wall. Thus, a water-tight seal was maintained when the head box was filled with water and the glass panels bulged slightly due to the hydrostatic pressure exerted by the water.

<u>Sluices</u> - Because the forces involved in the flow through the sluiceways are viscous as well as gravitational, it is impossible to obtain flow in the model that is completely similar dynamically to that in the prototype - to do so would require maintaining both the Froude number and the Reynolds number a constant from the model to the prototype. Experience with closed conduit models and their prototypes has shown, however, that the discrepancies indicated in the theory of similitude are not actually as serious as they appear. If the absolute size of the sluices is kept reasonably large, as they intentionally were in the Hirakud model, the agreement between model and prototype has been

found to be satisfactory. Tests on the Cherokee and Douglas Dam model and prototype sluices, constructed by the Tennessee Valley Authority, showed excellent agreement with regard to pressures, capacity, and performance at the outlet end. Some of the model data was obtained from a model sluice slightly larger than the Hirakud sluice and the remainder from a model somewhat smaller. Consequently, it is believed that the data taken from the Hirakud sluice model based on strictly geometric similarity will be entirely reliable.

The sluices were constructed of sheet metal to the shape shown in Plate 7 and Fig. 2. In the model, 4-1/2 sluices were included with the half-sluice being placed against the glass panel to allow observation of the flow within the sluice. A half-gate also was installed in this sluice.

It was necessary at this point to plan ahead so that the sluices discharging onto the service spillway apron could be modified easily and used for tests on the auxiliary spillway sluices which were to be tested later. Since the pressurized portion of the sluices was the same for both spillways it was constructed to the outlines shown in Fig. 2. The inlet was 16.5 ft wide by 30 ft high and the gate section was 12 ft wide by 20 ft high. The open channel or downstream portion of the sluices differed in the auxiliary and service sections however, and consequently the roof portion of the sluice downstream from the gate was made to resemble a sheet metal box into which suitable wood inserts could be fitted to obtain the desired shape. The roof inserts were made of mahogany which after being properly shaped were treated with applications of Phenoplast to prevent warping due to water-soaking. Since the barrel of the sluice was 12 ft wide and 21 ft high with a 6-ft-radius arched roof, also curved in the longitudinal direction, construction of the sluices using wood inserts was accomplished more quickly and economically than if sheet metal or plastic had been used.

One of the sluices was equipped with an outlet pipe 3/4-in. in diameter attached to the vent pipe which was carried out the sidewall of the test flume. Although the devices to measure the air demand were completed and ready for operation, air was not entrained by the flowing water so the air measuring devices were never used.

The sluice equipped to measure the air demand contained the piezometers necessary to determine the pressure distribution throughout the sluices. A total of 18 piezometers were installed along the centerline of the bottom of the sluice, 8 along the centerline of the sluice sidwall, and 7 along the junction of the sidewall and roof of the sluice inlet where minimum pressures were expected. The piezometer tubes were similar to those used in the spillway crest. In the sluices, however, the tubes were soldered in holes drilled through the sheet metal and then filed flush with the inside surface. The piezometers were connected to glass manometers outside the flume using gum rubber tubing of suitable diameter.

The lower portion of the steel framework of the model, the portion underneath the sluices, was embedded in a concrete block poured inside the test flume. The block was sealed to the flume floor and sidewalls and had sheet metal inserts in the top face to which the sluices could be secured by soldering (Plate 8). Metal tabs on the sluices were soldered to the metal inserts to properly align the sluices. The upstream and downstream spillway faces were then installed and the inlet and outlet ends of the sluices soldered to the sheet metal spillway faces.

Stilling Basin - The stilling basin of the service spillway is shown in Fig. 1. Since changes in the elevation of the apron were anticipated, the bucket was constructed of masonite and the supporting frames and the apron were constructed of wood to allow major rebuilding of the bucket and apron with a minimum of effort. The templets used to form the bucket and support the apron were constructed of 1-in. lumber. The apron supports were designed to take advantage of the fact that wood swells only a very slight amount in the direction of the grain. Thus, the apron did not become elevated due to wood swelling. The apron surface was 1/2-in. plywood while the bucket surface, which was curved, was made of flexible 1/8-in. tempered masonite.

The end sill used on the original apron was 40 ft high, trapezoidal in section, 10 ft wide at the top, and had a 1 on 2 slope on both the upstream and downstream faces (Fig. 1). It was constructed of 1/2-in. plywood, and treated with Phenoplast to reduce swelling and warping of the wood. Keyhole shaped slots were cut in the bottom of the sill and were used to facilitate moving the sill to different positions on the apron. Rows of round-head wood screws, screwed almost down to the apron surface, were placed at 6-in. intervals upstream from the original position. The round portion of the keyhole slots in the sill was placed over the heads of the wood screws and the entire sill was then slid downstream to lock the screw heads in the narrow part of the keyhole slot. The sill was thus anchored firmly but could be readily moved to different positions.

Equipment and Procedure for Testing the Original Model

Although it is of utmost importance that the flow boundaries of the model be constructed with extreme accuracy, it is of equal importance that the proper equipment be used with the correct procedure for making measurements of discharge, pressure, elevation, and erosion. Incorrect testing procedures may lead to data of questionable value. The following discussion pertains to the testing equipment and how it was used to obtain the data.

Measurement of Discharge - The water used to operate the model was stored in the sumps located below the floor of the laboratory. The water was delivered to the model through a 14-in. pipe from a 20-HP, propeller-type pump and through an 8-in. pipe from a 15-HP turbine-type pump. A 10-1/2-in. orifice plate was placed in the 14-in. line 18 ft from the pump, and a 6-in. orifice plate was used in the 8-in. line. Both orifice plates, a part of the standard laboratory equipment, had been carefully calibrated.

Discharges were determined from the differential heads indicated on the manometers connected to the 6-in. and 10-1/2-in. orifices. A water manometer was used on the 10-1/2-in. orifice and a mercury manometer on the 6-in. orifice. The differential head values obtained during a test were converted to discharge, using previously prepared rating curves and tables.

Measurement of Pressure - All pressure measurements were made using piezometers connected to open water manometers consisting of small diameter glass tubes. Readings on the manometers were made directly in feet of water, prototype. The same manometer bank was used to measure the pressure distributions both on the crest of the overfall spillway and throughout the air sluiceway. The piezometer openings on the spillway crest, as well as in the air sluice, were connected directly by independent tubes to the manometer board so that the pressures at all piezometer locations could be measured quickly and so that the pressure distribution, indicated by a group of piezometers, could be seen at a glance. The piezometers were made of 3/32-in. O.D. brass tubing having a 1/16-in. hole and the manometers were made of 1/8-in. I.D. glass tubing. These were connected with 1/4-in. O.D. gum rubber tubing. When the piezometers were installed in the model, great care was taken to make the surface surrounding the piezometer holes very smooth so that no burrs or other irregularities would create local conditions which would not be representative of the true pressure distribution. In addition, the piezometer hole itself was carefully reamed to remove the sharp edges and any projections.

The manometer fluid was made of a solution of 90% water, 5% Aerosol, and 5% fluorescein dye. The fluorescein was added to the water to give it color to make the meniscus more readily visible, and the Aerosol was added to relieve the surface tension and resulting capillary rise which would have otherwise occurred in the 1/8-in. glass tubes of the manometer board. To eliminate the possibility of an air lock in the tubes, the systems were primed by flushing with manometer fluid from the open end of the manometers through the piezometer holes. After priming, the zero reading for each manometer was determined by marking on the manometer tube the elevation at which the fluid came to rest after the excess fluid had drained out of thepiezometer holes.

The procedure for making pressure measurements was to set the desired flow in the model and allow the flow to reach equilibrium. Each piezometer assembly was then primed and the fluid in the manometer allowed to reach equilibrium. The height of the meniscus above the zero mark was measured using a special scale marked in prototype feet. These data were tabulated on a standard laboratory data sheet together with discharge, headwater elevation, tailwater elevation, and other necessary information.

Measurement of Water-Surface Elevation - The determination of the water surface elevation at a given point was accomplished by two different methods, depending upon the accuracy required. For precise determinations of headwater elevations, a hook gage operating in a gage well was used. The gage well was a 4-in. diameter sheet metal cylinder with a length of 1/2-in. I.D. hose to conduct water from the head box to the gage well. The hook gage and well were fastened to the head box, using separate mountings so that the gage point could be moved vertically along the axis of the well. Because of its simplicity and ease of operation, the Lory-type gage was used for all precise measurements of water-surface elevations. The shaft of the gage was made of 3/8-in. square brass tubing graduated in 100ths of a foot. It was adjusted by a rack and pinion movement and could be read to 1000ths of a foot by using the attached vernier. If fluctuations in the gage well were evident, several readings were taken over a period of time in order to determine the average value.

For determination of the tailwater elevation, a staff gage was attached to the glass wall of the test flume at a point 520 ft downstream from the face of the dam. The staff gage appears at the downstream end of the glass panel in Plate 1. Using this gage the tailwater could be set quickly and accurately by adjusting the tailgate.

In addition to the measurements described above, it was necessary to provide a method for measuring the water surface profile throughout the length of the hydraulic jump in the stilling basin. Also, it was necessary to provide a method for determining the amount of erosion and bed movement during and after each test. Both needs were satisfied by ruling a grid system on the glass walls of the test flume marked in 10-ft vertical increments and 20-ft horizontal increments, extending from the downstream face of the dam to the end of the glass-walled section. The grid is shown in the photographs of model performance.

Measurement of Erosion - Although it was expected that the bed of the river downstream from the service spillway would be composed primarily of solid rock, a prototype size of 12-in. bed material was assumed as a basis for the model studies. The bed material used in the model was screened gravel of 0.056-ft mean diameter. On a strictly geometric scale basis this represented loose rock with a mean diameter of 2.24 ft. According to the velocity ratio equation

$$(w/v)_m = (w/v)_p$$

this corresponded to 1.0-ft diameter bed material in the prototype. The use of this parameter is based on the work of Krumbein (1) and Rouse (2) 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 w/v was kept a constant from model to prototype. In this ration w is the fall velocity and v is the characteristic velocity of flow.

To make an erosion test for any given flow condition, the river bed of the model first was molded to the proper shape. Next, the river channel and the stilling basin were slowly filled so as not to distrub the bed material, and the headwater, tailwater, and discharge were set. Each test was run for 20 minutes so that all measurements would be comparable, and the model was then shut down and carefully drained. Sketches were made of the erosion profile as observed through the glass-walled channel and photographs were taken of the existing scour.

B. DISCUSSION OF TEST RESULTS FOR ORIGINAL MODEL

Since the sectional model represented only four and one-half of the 16 sluices in the service spillway, the testing procedure was of necessity arranged to cover as many combinations as possible of the open and closed gates in that part of the structure not modeled. For example, in a test for a particular gate setting and headwater elevation in the sectional model, the entire prototype spillway discharge might vary depending on how many other gates are considered to be open. Thus, for one setting of the headwater and gates, the operation of the model was observed for a wide range of spillway discharges by varying the tailwater from the minimum which would occur when only a few gates were open up to the maximum which would occur when all the gates were open. The tailwater discharge curve is shown in Fig. 3.

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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 #20.

In addition to varying the tailwater elevation, the model was also tested over the full range of headwater elevations from the minimum at Elev. 590 to the normal head at Elev. 625. The relation between headwater elevation and discharge is shown in Fig. 4. Since the sluices in the prototype will all be opened before any overfall spillway discharge occurs, and since the spillway gates will be operated either fully open or fully closed, the curve in Fig. 4 shows the combined overfall spillway and sluice discharge for the entire range of headwater. The relation between headwater and discharge for the overfall spillway only is shown in Fig. 5. Sluice discharges are shown in Fig. 6.

During the tests the flow patterns throughout the structure were noted and recorded, pressures were measured in the parts of the structure believed to be critical, and the spillway and sluice capacities were determined. The performance of the stilling basin was evaluated by means of erosion tests and measurement of waves and surges. Modifications to the stilling basin were made to test the shape and size of the end sill to determine the proper length of the stilling basin and to evaluate the apron performance at three different apron elevations. Each modification was tested with three different tailwater elevations which were selected as representative of the conditions to be expected over the full tailwater range. A stilling basin design recommended for field construction, was developed and tested and its performance is discussed. No modifications in the spillway crest, gates, or piers was found by the laboratory to be necessary. The sluices, however, were modified considerably but the results are too extensive to be summarized here. Therefore, Chapter III is devoted to this problem.

For the purpose of discussion, the tests are divided into three separate categories; the overfall spillway, the sluices , and the stilling basin.

Overfall Spillway

The performance of the crest section of the spillway, including the spillway piers, the gates, and the spillway face down to the sluice openings, presented no problem. This portion of the structure performed satisfactorily in that the capacity of the spillway was adequate, pressures on the crest were all above atmospheric, the piers introduced no unusual flow patterns, and the flow patterns upstream from the structure and on the spillway crest were considered to be as smooth and uniform as necessary. A discussion of these phenomena is given in the following paragraphs covering the flow pattern, the pressure distribution, and the spillway capacity.

Flow Pattern - The model was tested in a flume with parallel side walls. Since topography at the reservoir shore line could not be reproduced, it was impossible to make a full study of the approach conditions. In the model, however, there was no indication of a problem involving the water approaching the spillway, even for the maximum discharge at headwater elevation 630. As the flow entered the spillway the piers caused a slight contraction in the flow, resulting in a slightly depressed water surface close to the piers but this was of a minor nature and was considered to be unimportant (see Fig. 7).

On the downstream face of the spillway just below the piers, the flow from adjacent bays expanded laterally and the two nappes came together about 30 feet from the downstream end of the pier causing a fin to be formed. The fin was about 4 feet high, maximum, and although in the model it created very little disturbance, it would no doubt be more noticeable in the prototype. Some spray would be formed but this would probably be of no consequence. Flow downstream the fin was smooth and uniform on the spillway face and entered the spillway bucket in a satisfactory manner.

Before the model was tested it was believed that some type of deflector or "eyebrow" might be necessary to prevent the spillway flow from dropping into the sluice outlet openings in the face of the spillway. In operation, however, the model showed that since the total flow from the sluices is so much greater than the total flow over the spillway crest, very little undesirable effect is obtained that could be corrected by the use of eyebrows. Since it is planned to operate the overfall spillway only after the sluices are in operation, eyebrows were not considered necessary.

The spillway flow in passing over the sluice openings did, however, cause a back pressure at the outlet end of the sluice which tended to make the sluice flow full at the outlet. Venting in the upper end of the sluice had little, if any effect on the flow pattern at the intersection of the two flows. Flow conditions in the sluices are discussed in greater detail in a following section of the report.

Pressure Distribution - Pressures on the spillway crest were measured for a discharge of 5180 cfs per bay with headwater Elev. 630 and the spillway gates fully open. The location and elevation of the piezometers and the pressures measured are shown in Fig. 7. Pressures on the spillway crest were found to be above atmospheric for all flow conditions. The minimum pressure measured was about one foot of water above atmospheric, providing a safe margin against cavitation which might be caused by local irregularities in the prototype surface. The shape of the crest had been designed to approximate the underside of a nappe flowing over a sharp crested weir with headwater at Elev. 630 and for a 1.0 ft thick jet with headwater at Elev. 625. The crest pressures are considered to be amply safe and no difficulty should occur due to undesirable pressures.

Spillway Capacity - The capacity of the overfall spillway was determined for headwater elevations up to the maximum of Elev. 630. The curve of discharge versus reservoir elevation is shown in Fig. 4. At maximum headwater the discharge was 5180 cfs per bay and at headwater elevation 625 it was 2980 cfs per bay which compared favorably with the discharges intended by the designers. Fig. 5 also shows the coefficient of discharge C plotted against reservoir elevation for the same range of discharges. This curve shows that the coefficient increased from 3.51 at headwater Elev. 620 to 3.69 at headwater Elev. 630, indicating a favorable crest design. Because of the greatly expanded abscissa scale, the discharge coefficients may be used to compute discharges for various reservoir elevations with more accuracy and consistency than they may be taken directly from the discharge curve. Discharges may be calculated from the equation $Q = C L H^{3/2}$ where C is the coefficient of discharge, L is the length of the crest measured between piers and H is the difference in elevation between the reservoir water surface and the spillway crest.

Due to the fact that the spillway crest gates were intended always to be operated fully open, no tests were run with the gates partially open.

Sluices

The model was planned from the beginning to consist of four and one-half bays so that the half-sluice could be constructed against the glass wall of the test flume for observation of the flow conditions inside the sluice (Plate 7). Details of the sluices as tested are shown in Fig. 8. The half-sluices was equipped with a half-gate in order that flow during the opening operation could be observed and so that flow conditions at the sluice outlet could be studied when only the spillway was operating. Piezometers had been installed in critical areas in one of the full sluices as was an air-measuring device to measure the air demand for various operating conditions.

In making a complete test the flow pattern was observed and noted, the pressures measured and recorded, the capacity of the sluice determined, and the air vent operation observed.

Flow Pattern - Downstream from the gate the sluice was designed to operate as an open channel with an air space between the roof and the surface of the flowing water. Near the outlet, however, the clearance between the surface of the water and the roof of the sluice was not adequate and the space was filled periodically with a mixture of air and water. The result was a surging action within the sluice which was especially pronounced when the height of the tail-water was such as to partially submerge the sluice outlet. This action was even morenoticeable when the spillway was operating. Under these conditions the sheet of water from the crest added to the congestion at the outlet end of the sluice. The air vents did not prevent the sluice from filling at the lower end and attempts to change the flow pattern by inserting vent tubes in the lower region met with little success. The general action at the sluice outlet was considered undesirable and a revision in the entire sluice design, which is described later, was made to improve the performance.

Pressure Distribution - To determine the pressure distribution in the sluice inlet transition piezometers were installed along the upper corner of the pressurized portion of the sluice because previous experience in similar tests had showed that this was the most critical area. If pressures along the line of intersection of the sidewalls with roof were above atmospheric it would be almost certain that pressures in any other part of the entrance would also be above atmospheric. Additional piezometers were installed, however, to check the pressures throughout the sluice to be certain that the lowest pressure area would be measured.

The test results showed the correctness of the original concept that the lowest pressures occurred along the upper corner. In designing the sluice entrance the space limitations were so severe that the transition from the upstream face of the dam into the barrel of the sluice had to be very rapid. This made the radius of curvature relatively small which, combined with the rapid acceleration of flow in the entrance, caused the pressures to be reduced considerably below atmospheric. Fig. 2 shows graphically the pressure distribution along the side wall of the sluice at Elev. 520 along the centerline of the bottom, and in the upper corner of the entrance.

Along the side centerline and along the bottom of the sluiceway, the pressure distribution was found to be greater than atmospheric throughout the inlet transition. Downstream from the gate in the open channel portion of the sluice where the bottom curvature became pronounced, the pressure along the bottom centerline dropped to 14 feet less than atmospheric when the tailwater was at Elev. 509. With the tailwater raised to Elev. 539, the minimum pressure was increased to 12 feet below atmospheric. Although these pressures are not in the cavitation range they are sufficiently low that local irregularities in the prototype concrete surface might produce cavitation -- especially with water velocities approaching 90 feet per second.

Of equal concern was the pressure distribution along the upper corner of the sluice in the entrance transition. Here, pressures were below atmospheric for a considerable distance with a minimum pressure of minus 23 feet of water occurring at piezometer No. 15 (Fig. 2). There is little doubt that a pressure of this magnitude would produce cavitation in the prototype, resulting in pitting of the sluice inlets.

Because the sluices had been designed using the best data available and corrective measures could not be prescribed without extensive investigation, means for correcting this condition were discussed with the designers and a special model study, described in Chapter III, was initiated.

Sluice Capacity - The capacity of the sluices was determined only with the gates fully open because discharges with the gates partially open were not contemplated or necessary in the prototype. Discharges were measured for headwater elevations between Elev. 590 and Elev. 630. The resulting curve, discharge versus reservoir elevation, is shown in Fig. 6. At Elev. 625 the discharge for a single sluice was 17,180 cfs and at Elev. 630 it was 17,520 cfs which compared favorably with the discharge of 14,500 cfs used by the designers in preliminary estimates. Consequently, the sluices were sufficiently large to supply the desired discharges. The coefficient of discharge plotted against reservoir elevation is shown in Fig. 6. Discharges may be computed for any reservoir elevation from the equation

$$Q = C_d A \sqrt{2gh}$$

where C_d is the coefficient of discharge, A is the area of the sluice immediately upstream from the gate, and H is the head on the sluice measured from the reservoir water surface elevation to the centerline of the sluice (free discharge). Combining the constants for this sluice the equation reduces to

$$Q = 1928 \quad C_d \sqrt{h}$$

Air Demand - The model was originally designed and constructed so that the quantity of air necessary to completely ventilate the sluice during operation could be measured. Tests for a full range of headwater elevations, however, showed that there was no air demand through the vents. Instead of a continuous flow of air into the sluice as might be expected, the surging action at the end of the sluice, previously described, caused a similar action in the vent. Air was intermittently drawn in and out of the vent. At times water rose in the vent and was then drawn back into the sluice to allow passage of air for only a short time. No measurements of any value could be made and consequently no data are presented. Pressurizing the sluices, as described later, made further studies of the vents unnecessary.

Stilling Basin

The service section of the prototype spillway consists of 17 crest bays and 16 sluices similar to the four and one-half bays built in the model. The remainder of the spillway in the prototype structure extends to the left of the service section. A training wall is to be constructed between the service and the auxiliary spillway sections to prevent lateral flow between them. In the vicinity of the training walls and the end of the service section apron, it is believed there are problems to be overcome because of lateral currents introduced in the lower channel as a result of various combinations of open gates and sluices. Since the model was of the sectional type, the information which could be gained from it was limited to two-dimensional flow patterns. Lateral movement of the water or the bed material could not be fully or accurately studied nor could accurate predictions be made of the effects of unsymmetrical gate or sluice arrangements. These conditions would have to be determined in a model consisting of the entire section of the spillway and a considerable area above and below the dam. Thus the results of the tests on this model necessarily assume that the adjacent spillway bays and sluices, which were not constructed, would have no lateral effect on those operating and being tested.

The apron of the service section of the stilling basin was at Elev. 452 and extended downstream a distance of 420 ft from the axis of the dam. At the end of the apron a sill 40 feet high with an upstream slope of 1 on 2 was installed so that it could easily be moved upstream to determine the effect of apron length.

Although at the time these tests were made it had already been decided to pressurize the sluices, the sluices were not rebuilt in the model since it was not considered necessary. The open channel sluices carried the proper amount of water at the proper velocity and introduced the flow onto the apron in the proper direction. Action on the apron, therefore, was the same as though the sluices had been pressurized, and the stilling basin tests were considered valid as though they had been made according to the later recommended design of the sluices.

Tests were made to evaluate the overall performance of the stilling basin, apron, and end sill. To aid in determining the relative value of a particular design, the action was photographed and the water surface profiles were sketched using as a guide the grid lines which had been ruled on the glass panels of the test flume. The erosion in the river bed was measured and the height of waves and surges in the lower channel were observed and recorded. The tests were made over a considerable range of discharges and tailwater elevations and, to cover any irregularities in the tailwater curve, tests were made for elevations both 10 ft above and 10 ft below the values from the curve.

General Performance - For maximum and near maximum discharges, the action on the apron was extremely violent. This was due to the high discharge per foot of width of apron, being 970 cfs per ft, combined with the relatively high velocity of flow entering the basin, being about 90 feet per second. The action was further intensified by the concentration of flow at the sluice outlets which was 17,000 cfs in a width of 12 feet or 1,420 cfs per ft of width. These concentrations are considerably higher than those usually found in even the largest structures and indicate a basic reason for model studies to obtain a satisfactory stilling basin.

In spite of the undesirable violent action on the apron which resulted in high waves and surges in the lower channel (see Plate 12), the operation of the basin indicated that the structure could be made to perform satisfactorily. A roller or modification of the hydraulic jump was formed on the apron which, regardless of the tailwater elevation could not be swept downstream. This was due in part to the high end sill. The sill, however, directed to the surface the high velocity jet flowing along the bottom of the stilling basin. Because the energy of this jet was only partly dissipated on the apron, a high boil was formed just downstream from the sill. The resulting high surface velocity together with the boil, created unusually large waves and surges in the channel downstream so that the water surface in the entire test flume was extremely rough.

The high-velocity jet, after rising to the surface downstream from the end sill, plunged again toward the river bottom causing excessive erosion of the stream bed, particularly for the lower tailwater elevations which might occur for a given discharge.

For all tailwater elevations, the toe of the jump or roller formed against the sluice outlets causing a slight backwater effect and corresponding reduction in discharge. Although this action probably helped to break up the sluice jets as soon as they emerged and aided in dissipating energy on the apron, the surging action created in the sluice was considered undesirable.

<u>Waves and Surges</u> - Aside from the creation of spray, surges and waves on the apron itself were felt to be of no particular concern although they did cause some surging action within the sluices. Of greater concern in these tests were the waves in the downstream channel. Maximum and average heights of the water surface disturbances in the lower channel, as far as 200 ft from the end of the apron were used to determine the relative value of a particular design. In addition, the wave heights were an indication of the type of flow to be expected in the prototype structure.

Wave and surge heights were measured by alternately observing the maximum crest and minimum trough of the waves and surges for a period of about one minute. The waves and surges thus measured were felt to be excessively large (see Plate 12), even taking into account the concentrated discharges entering the apron. It was also believed that they should be substantially reduced before the performance of the apron could be considered satisfactory.

For headwater Elev. 625 and tailwater Elev. 550, as may be seen in Plate 12 and Fig. 9, the water rose over the end sill proper in the form of a high wave to approximately Elev. 557. At a distance of about 100 ft downstream from the sill, it descended to Elev. 545. This resulted in a wave 10 ft high from crest to trough. Although upstream from the sill (Fig. 9) the water surface was quieter, it none the less had waves of approximately 10 ft in height. The outlet of the sluice was submerged with approximately 10 ft of water.

For a tailwater elevation 15 ft lower, the surface of the water upstream from the sill was disturbed and churning somewhat more violently. Over the sill the waves extended to a height of about Elev. 550 and then dipped 100 feet downstream to about Elev. 535 thereby giving a wave height of 15 ft from crest to trough.

Under conditions of low tailwater at Elev. 520, the water was shurning most violently. Upstream from the sill, the sluice outlet was more than half submerged with the water level at any given point fluctuating nearly 20 ft over a short interval of time. Over the sill proper the water rose to a height of 35 ft above that in the stilling basin to Elev. 545 and then dropped below Elev. 520 approximately 60 ft downstream from the sill. This resulted in a wave height of 20 ft.

The general action and appearance of the water flowing over the sill gave the impression of a secondary dam with the energy of the flow being only slightly reduced from that which entered the stilling basin. As a result, the nappe-like jet coming over the sill plunged into the tailwater downstream to cause waves and disturbances in general which were most unsatisfactory.

Although the surfaces, as represented in Fig. 9, appear to be rather smooth this is due to an averaging effect. The photograph in Plate 12 shows to some extend how violent and irregular the surface was and also the fact that breakers were leaping into the air to a height of 30 to 40 ft.

The spray resulting from such action even in the model was considerable, rising to the height of the dam. In the prototype, however, the quantity of spray would be increased many times due to the greater forces tending to break up the prototype flow and the greater magnitude of the air currents surrounding the dam. This spray would no doubt be carried to great heights completely saturating large areas near the dam.

It was evident, then, that the original design of the end sill was not satisfactory from thepoint of view of the excessive waves and surges within the stilling basin, over the sill, and downstream from the sill. The sill was not only too high, but it also projected the high-velocity water upward at too steep an angle.

It was believed that revisions in the shape of the end sill could be made to improve the performance but it should be kept in mind that even with the most efficient end-sill design, waves and surges of considerable size can be expected. From observations of the general flow pattern, however, it was believed that improved performance could be obtained.

Erosion Test - In making erosion tests, the erodible bed was molded to the profile shown in Fig. 9 and the lower channel was filled slowly to prevent premature movement of the erodible material. Then the tailwater had reached the desired level the flow through the model was gradually increased. The discharge and tailwater were then set exactly and the tests allowed to run for 20 minutes.

After several preliminary runs varying in time from 15 minutes to 2 hours, it was decided to use 20 minutes as the standard running time since most of the measurable erosion occurred within this time. Although after 20 minutes, the erosion continued according to a logarithmic law, as established by Rouse (1)

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1. Rouse, Hunter Criteria for similarity in the transportation of sediment. Proceedings of Iowa Hydraulics Conference (1939). State University of Iowa Studies in Engineering Bulletin #20.

and Doddiah (1), it was believed that comparative values could be obtained so that relative amounts of erosion could be evaluated. Following the logarithmic law, these preliminary erosion measurements showed that the equation $D = m \log t + b$ gave approximately the relationship between depth of erosion D and time t after the first 15 minutes of running. The constants m and b depended upon the quantity of water flowing, the depth of tailwater, and the relative geometry of the bed material and the stilling basin.

For the tests on the original design, the depth of erosion may be seen in Plate 12 and Fig. 9. With the headwater at elevation 625, the erosion downstream from the sill was approximately to Elev. 478 for tailwater Elev. 550, and Elev. 468 for tailwater Elev. 520. Before each test, the bed material downstream from the sill was leveled at Elev. 492. After a 20 minute run, however, a very large quantity of material had been removed -- rough computations indicating this to be in the neighborhood of 2,000 cubic yards per yard of width of channel.

Although the material in the prototype may be larger and more securely held in place, the erosion which would result would be similar to that obtained in the model but would take a greater length of time than indicated in the model. As may be seen in Plate 12 and Fig. 9, however, the erosion was located far enough downstream so that the structure itself was not endangered for the conditions tested. The amount of erosion being considered undesirable, it was believed that modification of the apron and sill would reduce the erosion and improve the performance.

Recapitulation - From the test made on the stilling basin of the original design of the service section, it was concluded that the performance was unsatisfactory but could be improved. The violent action on the apron formed excessive waves and surges and caused considerable erosion. The roller action on the apron moved upstream to cause surges in the sluices and the general appearance of the entire flow pattern was unsatisfactory. It was believed that by modifying the height, shape, and location of the end sill, and by making tests on aprons at other elevations, satisfactory operation could be obtained.

C. RECOMMENDED REVISIONS

Overfall Spillway

Although there was a depressed water surface close to the piers caused by contractions in the flow, the effect was considered negligible and not worty of attempts to improve the flow profile. Other flow disturbances on the overfall spillway were also considered to be minor in nature and consequently the original design of the crest structure is recommended for use without change.

Sluices

The performance of the sluices was considered to be unsatisfactory from two viewpoints. First, the pressures in the upper end were in the cavitation

1. Doddiah, D. Comparison of scour caused by hollow and solid jets of water. Thesis for Master of Science in Irrigation Engineering. Colorado Agricultural and Mechanical College. December 1949.

range and second, the open channel type of sluice introduced problems at the downstream end of the sluice that could not be readily solved. Consequently, it was decided to use a pressurized type of sluice and to redesign the sluice inlet to obtain more nearly atmospheric pressures throughout the sluice. It was felt that several trials would be necessary to obtain a satisfactory entrance or bellmouth shape because space limitations prevented use of some of the most effective modifications to the inlet.

It was not possible to move the sluice gate downstream to increase the radius of curvature in the inlet transition. This would have placed the operating gallery too near the face of the spillway. Also, in the interests of economy the designers were attempting to keep the sluice gate and the emergency gate to a minimum size.

It was decided that a special study should be made of this problem on an entirely separate model where only one sluice was modeled and where changes in inlet shape could be easily and quickly made. An extensive study of the sluice design was eventually made and is reported separately in Chapter III.

Stilling Basin

It was believed that the main cause of the high boil, the excessive wave heights and surges, and the deep erosion found for the original design of the stilling basin was caused by the excessive height and steep upstream slope of the end sill. Therefore, tests were first made to determine the effect of the end sill design on the performance. Using the best sill design, the effect of shortening the apron was then investigated by testing aprons which extended from 300 ft to 420 ft downstream from the axis of the dam. Finally, the effect of apron elevation on the performance was evaluated by making tests with the apron at Elev. 452, 462, and 472.

Sill Design - In order to have a standard for comparison of the effectiveness of the various sills and to determine how much effect the sills had on wave heights it was decided to first make tests with no sill on the apron. As may be seen in Fig. 9, the waves were much smaller and the surface smoother in general than was found with the 40 ft sill. Under all conditions of tailwater elevations, the surface of the water in the stilling basin gradually rose to the downstream tailwater height. The sluice outlet was somewhat submerged for tailwater Elev. 564 and Elev. 540 but for tailwater Elev. 520 it was relatively unsubmerged. It should be noted in passing that the effect of submergency for tailwater Elev. 564 was to reduce the discharge through the 4-1/2 sluices in the model approximately 6,800 cfs.

Downstream from the apron the erosion which resulted from having no sill was appreciable. In the case of minimum tailwater, the erosion was to Elev. 426, and for maximum tailwater the erosion was to Elev. 436. Maximum erosion in both cases occurred approximately 80 ft downstream from the end of the apron. As may be seen in Fig. 9, the erosion immediately downstream from the apron was to a depth nearly 20 ft below the surface of the apron which would not doubt endanger its safety. It was not expected to use the apron without an end sill. Nevertheless the test serves as a reference guide for the magnitude of erosion, the flow patterns, and the wave heights within and downstream from the stilling basin.

Sill Revision No. 1 was made as a result of observing the flow, erosion and wave patterns evident in the original design. The sill was lowered from 40 ft to 20 ft and the upstream slope was reduced to 45 degrees (see Fig. 9). Since the jet leaving the original sill was completely unbroken as it descended into the tailwater downstream from the sill it was decided to use in the first revision a dentated sill. The dentils had vertical upstream faces, were 20 ft wide, and occupied 50% of the upstream face area as shown in Fig. 9. The dentated sill was intended: (1) to direct the flow into the lower channel at a flatter angle; (2) to spread out the boil over the end sill; and (3) to reduce the violence of the current or ground roller which plunged back toward the river bed.

Tests indicated that the first revision was a distinct improvement. Although the water surface was still quite irregular, it had fewer and smaller irregularities than the original design. As may be seen in Fig. 9, the maximum height of wave in the stilling basin was approximately 10 ft for tailwater Elev. 550. Under these conditions the sluice outlet was submerged by about 10 ft. Directly above the sill the surface rose in a wave approximately 8 to 10 ft in height. Due to the dentated sill, the wave descended on the downstream side with its energy clearly reduced. This resulted in much more desirable downstream conditions. With tailwater lowered to Elev. 535 and Elev. 520, the water surface in the stilling basin gradually rose from a minimum near the sluice outlet to a maximum over the sill. For tailwater 535, the wave over the sill was approximately 10 ft high and for tailwater Elev. 520 the wave was less than 10 ft. Within the stilling basin the action was in general less violent, yet the energy dissipation appeared to be more complete. Although the upstream slope of the sill was flatter than that of the original, it did not adversely affect the sweep-out characteristics of the basin. For all discharges the tailwater could be lowered below any possible prototype elevation without causing the jump to sweep out.

The downstream erosion resulting from sill revision No. 1 was much more desirable than experienced with the original design. Immediately downstream from the sill, the bed material was carried towards and held against the downstream face of the sill by the ground roller, whose intensity had been reduced by the dentated sill. In this case, the maximum erosion for minimum tailwater at Elev. 520 was to a depth of Elev. 463.

It appeared that (1) the sill could safely and effectively by lowered from 40 ft high to 20 ft high, (2) flattening the upstream slope of the sill resulted in better action and performance, and (3) placing dentils on the sill had the effect of breaking up the jet over the sill and reducing the boil in the tailwater downstream.

It appeared, however, that the action and performance of the basin could be further improved by reducing the slope of the sill even more. Tests were next made on a sill with a still flatter slope.

Sill Revision No. 2 was of the same general design as revision No. 1 except that the upstream face of the sill had a slope of 30 degrees, rather than 45 degrees. This sill may be seen in Fig. 10.

In operation, the sill reduced the surface disturbances, and thereby improved the performance of the basin. The height of the wave directly above the sill was reduced from 10 ft in revision No. 1 to approximately 4 ft in revision No. 2. The waves and surges within the stilling basin (still approximately 10 feet in height) were not materially affected -- the action there being somewhat independent of changes in sill design. Downstream from the end sill, however, the height of the waves was materially reduced. This was due primarily to the flatter angle of projection of the water as it left the sill.

As also may be seen in Fig. 10, theerosion downstream from the sill in revision No. 2 was to a maximum depth of approximately Elev. 463. This was for minimum tailwater Elev. 520. Because the direction in which the jet left the sill was more nearly horizontal in revision No. 2 than in revision No. 1, the maximum erosion for revision No. 2 occurred closer to the end sill than for sill revision No. 1. For revision No. 1 the maximum depth of erosion occurred from 60 to 100 ft downstream from the end sill whereas in revision No. 2 it occurred, in all cases, approximately 50 ft downstream. At a distance approximately 150 feet downstream, however, there was much less erosion for revision No. 2 than in revision No. 1. Although the extent or depth of erosion in either case was not of a serious nature it appeared that from the overall point of view the erosion resulting from revision No. 2 was more desirable than that found for revision No. 1.

After a thorough comparison of the action resulting from revision No. 1 with that from revision No. 2, it was decided that the slope of 30 degrees was most desirable and that a flatter slope would probably result in moving the point of maximum erosion further upstream to a point too close to the sill for safety. This general cross-sectional sill shape (see Fig. 10) was therefore decided upon as the most desirable for further studies.

Sill Height - In order to determine the effect of the size of the end sill, it was decided to test dentated sills patterned after revision No. 1 both 40 feet high and 10 feet high (see Fig. 10).

Sill Revision No. 3 had a height of 40 feet, and may be seen in Fig. 10 and Plate 13 together with the general flow pattern which resulted. As found for the original sill, waves were generated over the sill and extended into the tailwater downstream with a variation in surface elevation of 20 to 30 feet. This extreme condition (see Plate 13) occurred for tailwater Elev. 520. For tailwater Elev. 535, the wave directly above the sill was only 10 ft high, while for tailwater Elev. 550 the change in water surface was even less. In the stilling basin proper, the action was somewhat more violent than for the 20 ft sill. This is probably due to the fact that immediately upstream from the end sill the water was deeper than existed with the 20-foot sill.

The erosion pattern resulting from revision No. 3 was not materially changed from that in revision No. 2. For tailwater Elev. 520 the erosion was to Elev. 464, which was approximately the same as that in revision No. 2 (see Fig. 10).

In light of the foregoing discussion, then, it may be said that the 40 ft sill showed no improvement over the 20 ft sill of revision No. 2. In fact, the waves were more violent and it was concluded that this sill was not suitable for prototype construction.

Sill Revision No. 4 was non-dentated and was 10 feet high with a 30° upstream slope. To test this sill it was necessary to mold the river bed at Elev. 462. As may be seen in the photograph of Plate 14, the action within the stilling basin was extremely violent. The amount of energy dissipation which occurred in the basin from the time the flow entered the stilling basin to the time it came to the end of the apron was considerably less than occurred for revision No. 2. This can be seen in two ways; first by Plate 14, a photograph of the action in revision No. 4, and second, by comparing the depth of the water in the stilling basin immediately upstream from the end sill shown in Fig. 10. In revision No. 4, for tailwater Elev. 520, the water surface immediately upstream from the end sill was approximately Elev. 513, whereas for revision No. 2 the surface was Elev. 522. A similar reduction in flow area existed for the other tailwater elevations tested. At tailwater Elev. 550 the water surface just upstream from the sill was approximately Elev. 540 for the 10 ft sill and Elev. 548 for the 20 ft sill. Furthermore, the stability of the jump was not satisfactory with the 10 ft end sill alone. A small lowering of the tailwater caused the jump to be swept from the apron.

Since the bed had been molded at Elev. 462 for tests on revision No. 4 the erosion downstream cannot be compared directly with that for the other sill tests but the erosion results give an indication of what would eventually occur in the prototype if a 10 ft sill was used. The maximum depth of erosion was to Elev. 452, whereas the minimum depth of erosion was approximately to Elev. 460.

Tests on the 10 ft sill in revision No. 4 showed that a sill of this height was too low to accomplish the intended functions of an end sill. It failed to provide sufficient guidance to the water and some of the high-velocity flow passed over the sill without being turned, causing considerable erosion of the river bed.

In summing up the test made to determine the effects of sill height, it was apparent that a sill 40 feet high was not necessary to hold the action on the apron for low tailwater conditions. Furthermore, the 40-foot sill created larger waves both above the sill and downstream from it (see Plates 12 and 13, and Fig. 9). For a sill as low as 10 feet high, on the other hand, the erosion and jump sweep out characteristics were more unfavorable than for the sill 20 feet high (see Figs 9 and 10, and Plate 14). The 20 ft sill provided the necessary protection to the river bed and at the same time reduced to a minimum the violent action on the apron.

The general shape of the dentated end sill used in these tests was chosen not by chance, but as a result of extensive tests made by the Tennessee Valley Authority in the Hydraulic Laboratory at Norris, Tennessee. These tests were made to find an end sill design for the Kentucky Dam spillway. The Kentucky apron is similar in many respects to the Hirakud apron and advantage was taken of the Kentucky Dam test to shorten the Hirakud testing program.

Sill Location - The end sill was located at the end of the apron throughout the tests of sill design as shown in Fig 10. This was done in order to compare various sill heights and shapes. Once the most desirable sill was determined, that in revision No. 2, it was important to determine whether a saving in the cost of the structure could be made by reducing the length of the apron.

Observation of the tests already made indicated that the apron might be shortened without materially increasing the undesirable effects of the flow leaving the apron. Accordingly, the apron length was reduced in increments and the water surface profile, wave heights, and downstream erosion were measured, using the sill in revision No. 2. The shortest apron tested extended 300 ft downstream from the axis of the dam.

For revision No. 5 (see Fig. 11), the apron length was reduced 10 ft; for revision No. 6, 20 ft; for revision No. 7, 40 ft; for revision No. 8, 80 ft; and for revision No. 9, 120 ft. Perhaps the most effective way to consider the significant effects of these changes in location is to study the progressive changes in the flow pattern within the stilling basin and downstream from the end sill and to study, in a like manner, the changes in erosion patterns as the apron was made shorter.

As may be seen from the flow profiles in Fig. 11, and Plates 15 and 16, the water surface within the stilling basin itself became more irregular and the action more violent as the stilling basin was shortened. In revision No. 5, 10 ft shorter than the original, the action was not materially different than when the apron length was reduced 20 and 40 ft (see Fig. 12 and Plate 17). However, it can be seen that the water surface 50 ft upstream from the end sill dipped considerably to later rise approximately 20 to 30 ft over the top of the sill itself. With the sill upstream still further, as shown in revision Nos. 8 and 9 (Fig. 12 and Plate 18), this dip in the water surface became quite obvious, amounting to 20 to 30 feet. Likewise, the rise in the water surface as it passed over the end sill was increased to approximately 20 to 30 feet.

This action took place because the length of the stilling basin was reduced to the extent that there was insufficient volume in the basin for energy dissipation. For revision No. 2, the length of the stilling basin was approximately 5 times the downstream depth of the water, whereas in revision No. 9, this was reduced to approximately 1-1/2 to 2 times the downstream depth -- thereby reducing the opportunities for the jet to become thoroughly diffused by the mixing action upstream from the sill. Particularly noticeable in revisions Nos. 8 and 9 was the jet-like profile which the water took as it passed over the end sill. Downstream from the sill the jet dropped 10 to 15 feet before it reached equilibrium at tailwater depth and produced sizeable waves downstream from the end sill.

It should be mentioned here that the water surface profiles shown in Figs. 11 and 12 show the average profile and do not indicate the rapidly changing water surface which occurred in the model. Even the photographs fail to show the dynamic conditions which prevailed. Thus, although the profiles and photographs may appear to portray relatively mild action, the true evaluation of the action can only be gained from observing the model in operation. Considering energy dissipation, action in the stilling basin, and water surface profiles upstream and downstream from the sill, it was apparent that shortening the apron as indicated in both revisions No. 8 and No. 9 could not be justified.

The erosion profiles which resulted from shortening the apron also indicated a limit for reducing the length of the apron. Although the depth of erosion was not affected in revision Nos. 5, 6, and 7, the erosion in revision Nos. 8 and 9 was carried to a depth approximately 4 ft deeper than for revision No. 2, as shown in Figs. 11 and 12, and Plate 18. In no case, however, was the erosion so severe as to warrant great concern.

As a result of the foregoing considerations, it may be said that revisions Nos. 5, 6, and 7 result in flow patterns and erosion conditions which could be accepted without question in the final design. Although revision No. 8 had waves of considerable magnitude, it is no doubt possible to use this sill location without danger to the structure proper. It must be remembered, however, that the waves carried on downstream were of such a height that severe erosion might occur.

Apron Elevation - Once the most desirable shape, size, and location of end sill was determined there still remained the possibility that the apron could be raised to some extent to decrease the cost of the stilling basin in the deep section. Consequently, it was decided to test aprons at Elev. 462 and Elev. 472 in the hope that one or the other would prove to be adequate.

To properly introduce the flow into the stilling basin, the sluices were modified as shown in Figs. 13 and 14, and Plate 19. The sluice outlet curves were made tangent to the stilling basin bucket which also was modified. At this apron height of Elev. 472, the 20 ft - 30° end sill used in revision No. 2, was again tested with aprons extending from 300 feet to 420 feet from the axis of the dam.

With the apron at Elev. 472 and the tailwater below Elev. 538 (see Figs. 13 and 14, and Plate 19), the action within and downstream from the stilling basin was extremely violent and unstable regardless of the length of apron. For headwater Elev. 630 and tailwater Elev. 558, the operation was satisfactory when the apron extended at least 340 ft downstream from the axis. With the sill 300 ft from the axis (see Fig. 14), waves were from 10 to 20 ft in height. With the tailwater at Elev. 538, reasonably good operation resulted when the apron extended 360 ft or more downstream from the axis but waves up to 30 ft in height were developed when the apron length was reduced. Although some of the surface profiles of Figs. 13 and 14 appear to be quite regular and smooth, it may be seen in Plate 19 that the surface was very irregular, containing many waves and surges.

The performance of the apron was unsatisfactory in any case, however when the tailwater was lowered to Elev. 525, the jump was completely swept from the stilling basin after degradation had taken place for approximately 30 minutes. Furthermore, for tailwater elevations below Elev. 538, the jump was not well formed regardless of the apron length and the jump never completely developed before the jet of water shot over the end sill with only a small part of its energy dissipated. This condition existed throughout all the tests made with the apron at Elev. 472. It became increasingly noticeable, however, as the apron length was reduced.

The sluice outlets were unsubmerged when the tailwater was at Elev. 529. When the tailwater was at Elev. 538, the degree of submergence varied from approximately 50% to 80% depending upon the apron length. At tailwater Elev. 558 the submergence was complete plus 2 to 15 ft greater. Thus, raising the apron would not in itself prevent the sluice outlets from being submerged.

The erosion patterns resulting from test with the apron at Elev. 472 did not reflect fully the poor appearance of the flow pattern. The greatest depth of erosion was only 20 ft below the initial surface of the river bed. Had there been a greater length of channel downstream and had the model tests been run for a greater length of time, it is probable that the erosion and general degradation would have been greater than shown. Sufficient data was obtained, however, to prove conclusively that the apron at Elev. 472 was unsatisfactory and would, in the prototype, have produced dangerous operating conditions.

Thus, the tests with the apron at Elev. 472 showed that poor operation in general could be expected but that fairly good operation for the upper ranges of tailwater could be realized. On this basis it was believed that lowering the apron to Elev. 462 might increase the tailwater depth sufficiently to produce satisfactory operation over the entire range of flow conditions. Since raising the apron would reduce the cost, it was decided to test the performance of an apron at Elev. 462. Accordingly, the apron was rebuilt and the sluices and spillway bucket again modified (see Fig. 15) in a manner similar to that shown in Plate 19.

Although this apron elevation produced flow patterns which were much improved over those at Elev. 472, it was still found, as may be seen in Figs. 15 and 16, that the flow pattern was too turbulent and unstable for safe use. The hydraulic jump in the stilling basin developed more completely than with the higher apron, but energy dissipation on the apron was still not sufficient to produce a smooth flow leaving the apron. In Fig. 15 a wave over the end sill is evident for all tailwater elevations and all apron lengths.

The erosion which resulted with the apron at Elev. 462 was not serious, extending to a maximum of only 20 ft below the initial surface at a reasonable distance from the end of the apron. Erosion, therefore, was not the major factor in the decision to favor a lower apron. The fact that the action in the stilling basin was extremely violent and the jump was on the verge of being swept off the apron, also influenced the decision to use a lower apron. With tailwater 10 ft higher than normal for any given test, the action was greatly improved and the flow leaving the end sill was of lower velocity and created less disturbance in the lower channel. Since these tests with 10-ft higher tailwater approximated those with the apron at Elev. 452, it was readily apparent that the lower apron was more satisfactory.

From all points of view, the tests on the aprons at three elevations showed conclusively that the apron at Elev. 452 was the most desirable. The measured data and analyses given in this report all point to the same conclusion. All the engineers who viewed the tests believed that the lower apron provided a better operating structure with an ample margin of safety, against indefinite and unforeseen factors.

Sill Pressures - It also was desirable for design purposes to measure the pressures on the dentated sill. A piezometer was installed on the top face of one dentil immediately downstream from the upstream face to determine whether cavitation pressures existed and a second piezometer was placed at the center of the upstream face of the dentil to measure the forces acting on the sill.

In Fig. 17 it may be seen that in the region where low pressures might be expected to exist (piezometer No. 1), the pressures were all well above atmospheric for all apron lengths and tailwater elevations. Pressures varied with the tailwater elevation and the length of the apron. For tailwater Elev. 550, the pressure varied from 60 ft of water for the apron extending 420 ft from the axis to 54 ft for the 300 ft apron. With the tailwater at Elev. 520, on the other hand, the pressure varied from 34 ft to 32 ft for the same range of apron lengths. This reduction of pressure with shorter aprons was probably due to the combined effects of the increased velocity striking the sill due to less dissipation of the energy, and the reduction in statis pressure above the sill as a result of a more incomplete jump development. In any event, however, the pressure is so much above atmospheric there should be no concern regarding cavitation on the end sill.

On the other hand, the impact pressures, measured on the upstream face of the sill, piezometer No. 2 in Fig. 17, substantially increased as the sill was moved upstream -- due, no doubt, to the greater velocity of the jet striking the sill. With the apron extending 300 ft below the axis, the pressure varied only a small amount, 110 ft to 116 ft, over the range of tailwater elevation. With a 420-ft apron however, the pressure varied from 73 ft for tailwater 520 to 101 feet for tailwater Elev. 550. This effect was no doubt a result of considerable variation in the height of the water over the sill caused by the variation in tailwater elevation.

The pressures which have been described indicated the order of magnitude of the pressures which may be expected in the prototype. The total pressures or forces on the sill can be estimated from these data for purposes of design.

Chapter III

SLUICE INLET MODEL

A. MODEL CONSTRUCTION AND TESTING

After consideration of the various methods of relieving the sub-atmospheric pressures found in the original design of the inlet transition for the sluices discussed in Chapter II, it was decided to set up equipment and apparatus to study the problem in a more fundamental manner. This was necessary because the original inlet design had been based on the best available data and no method for obtaining a satisfactory entrance shape was known except to develop the inlet curves by model tests. The testing procedure which was finally developed consisted of three stages. First, a rectangular orifice was provided with an upstream forebay apron and downstream floor. The side contractions were measured on the free jet issuing from the orifice. Second, sidewalls were built and installed to conform to the shape of the side of the free jet and the resulting water-surface profile was measured. Third, a roof having the shape of the water surface was installed and the pressures in an upper corner checked to determine the possibilities of cavitation.

Problems to Be Considered

The problem in general consists of determining the shape of a jet of water after it has passed through an orifice corresponding to the size of opening necessary on the face of the dam. Using profile curves obtained from the sides and top of the jet, an enclosing structure may then be fitted around the jet so that pressures very nearly atmospheric will be obtained throughout the entrance. The size of orifice must be such that the jet area will be equal to the gate area in the region where the prototype gate is to be located. Also, the capacity of the sluice must be sufficient to pass the proper discharge at the desired head. The problems are discussed in detail in the following paragraphs.

Entrance - In most model tests it is important to accurately reproduce the approach conditions upstream from the structure to be tested. In the case of the Hirakud sluice entrance the river bottom is generally at or near the level of the sluice floor, and major irregularities in topography will be leveled out. Therefore, the problem of reproducing entrance conditions is simplified and becomes one of producing a relatively quiet, non-turbulent condition in the pool upstream from the orifice plate which represents the face of the dam.

Free Jet - The item of major importance in connection with the performance of the sluice inlet model is the flow pattern of the free jet. To determine the shape of the free jet, it is necessary to measure the side and top contractions resulting from various head-water elevations with the two aforementioned approach conditions. Due to the fact that flow of the free jet through the orifice plate is subject also to the force of gravity, the jet surface is deformed before it reaches the end of the model floor. The extent of deformation depends upon the velocity of efflux. Thus, it is practical to measure the side and top contractions for a distance from the orifice plate only as far as the vena contracta of the free jet. Downstream from this point the problem is of a different nature and entrance data are sufficient until a satisfactory entrance shape has been found.

Enclosed Jet - The pressure distribution existing on the surfaces enclosing a jet indicate whether a satisfactory inlet design has been obtained. Inlet design can be considered satisfactory if (1) atmospheric or nearly atmospheric pressures prevail throughout the inlet, or if (2) the pressures in the inlet can be raised to or near atmospheric by slightly constricting the sluice outlet. As previously described, it is known that the critical area in which to expect dangerously low pressures is at the junction of the side and the top transition curves.

The object of the sluice inlet model studies is to determine the proper transition required to contract the water to specified cross-sectional dimensions of the sluice barrel. Both the opening in the face of the dam, which is covered by an emergency gate, and the sluice gate must be kept to the smallest possible dimensions consistent with a satisfactory pressure distribution. Limitations on size and length of the transition are inherent in the geometry of the structure and in the discharge requirements of the sluices. The problems encountered as a result of these limitations are discussed later as they occurred.

Design and Construction of Original Model

In order to develop a satisfactory design for the sluice inlets quickly and economically, a separate model consisting of one sluice was constructed. This 1:40 scale model was used to measure the profiles of a free jet issuing from an orifice of the desired size.

General Layout - As may be seen from Plate 20, the general layout of the model consisted of an entrance tank, an orifice plate, and a sluice floor with a corbel. The upstream face of the orifice plate represented the upstream face of the sluice.

The water for the model was supplied by the 8-in. pump and pipe line described in Chapter II, and the discharge was measured by an orifice meter in the line.

Entrance Tank - The tank which was used to accommodate the sluice model was available as part of the standard laboratory equipment. This tank had an inside diameter of 5 ft and was 6.5 ft high. Built into the tank was an opening in the side, approximately 2 ft by 3-1/4 ft, with a transition leading to the opening (Fig, 18). This opening was covered with the plate supporting the orifice.

The 8-in. supply pipe was carried over the top of the tank and discharged vertically downward near the tank floor. A lattice-type baffle was constructed in the tank to reduce the turbulence in the flow entering the tank. A rock baffle was also placed between the inlet pipe and the orifice in order to further reduce the turbulence and to insure uniform flow approaching the orifice. This baffle extended from the floor to the top of the tank and was made of 2-in. by 6-in. vertical wood members with 1/2-in., galvanized-wire mesh nailed to the edges. The frame was filled with river gravel varying in size from 3/4-in. to 1-1/2-in.

In order to determine the water surface elevation in the tank, a stilling well with a Lory-type hook gage was mounted on the side. The stilling well was connected to the tank with a flexible hose.

Orifice Plate - A sharp-edged orifice cut in a steel plate represented the opening in the face of the dam. When water was passed through the orifice at heads corresponding to those in the prototype, the shape of the issuing jet was an indication of the shape of the necessary inlet transition.

The support for the orifice plate was made of a sheet of cold-rolled steel 30-1/2 inches wide by 44-1/2 inches high by 1/4 inch thick (Fig. 18). A rectangular hole was cut in the plate to dimensions which were slightly larger than the maximum size of orifice expected to be tested. This plate was bolted to the flange of the tank transition. A smaller plate containing the orifice to be tested was bolted to the inside of the orifice support plate. Thus, the size of the orifice could be changed without a great amount of modification to the apparatus. The first orifice tested was 16.5 ft by 35 ft, prototype, and was a sharp edged rectangle cut in a plate and carefully filed to the exact dimensions.

Sluice Floor and Corbel - Because it was known that changes in the floor of the sluice or in the corbel would not be necessary, these features were permanently installed in the basic model. Thus a proper entrance condition was obtained and the jet was supported on the bottom to provide the necessary entrance similarity with respect to the prototype structure. The corbel, shown in Fig. 18 was built to exact dimensions and mounted on the inside of the orifice plate with screws. It was made of Honduras mahogany and extended horizontally about 4 inches beyond each edge of the orifice opening.

The sluice floor was also made from mahogany and attached to the corbel with dowel pins through the orifice plate. This floor extended from the orifice plate to a point downstream from the prototype gate section which was approximately the point of origin of the parabolic sluice floor.

As explained in Chapter II it was necessary to test the sectional model under both single and multiple sluice operation. Likewise, it was necessary to test the sluice inlet model under the same conditions. Dividing walls parallel to the direction of flow were placed upstream from the inlet model to represent the condition when adjacent sluices were open. When the walls were removed, greater contraction at the sluice entrance occurred and represented flow conditions when only a single sluice was open.

Sluice Side Walls - The sluice walls were constructed from mahogany and were shaped to conform to the profile of the side of the free jet or to a predetermined curve as explained later. The sidewall profile templet was first cut from sheet metal which was used for forming the wood. The sidewalls were placed in the model so that the upstream end of the curve was flush with the upstream face of the orifice plate.

Equipment and Procedure for Testing

The general procedure for operating the model and determining the inlet curves and shapes is described in the following paragraphs under the subjects of the free jet, the pressure, and the discharge.

Measurement of Free Jet - In order to measure the dimensions of the free jet at various sections downstream from the sluice inlet, it was necessary to devise a method by which accurate measurement could be made on both the side and the top contractions of the jet. As shown in Fig. 18, a guide block was made with 1/16-in. holes carefully drilled so as to ensure perfect parallel alignment. The measuring holes were spaced 1/2 in. from center to center in both coordinate directions. The wire point gage consisted of a 10-in. length of 1/16-in. brass welding rod which had been ground to a sharp point. The rod was held in place by light friction between the rod and the sides of the hole. Thus, the rod could be manipulated to obtain accurate measurements of the cross-sectional shape of the free jet at a number of points downstream from the orifice. All measurements were immediately recorded on a standard laboratory data sheet, together with other pertinent information such as headwater elevation and discharge.

Measurements of the free jet were taken for headwater Elevs. 590, 613, 625, and 630. The effect of changes in approach conditions, caused by multiple-sluice operation, was determined by installing wooden flow guides in the model. The guides produced streamlines similar to those occurring in the prototype when all sluices were in operation. To simulate single sluice operation, the wooden flow guides were merely removed. It was expected that a single sluice operation would produce the critical condition of maximum contraction. In this case the streamlines were allowed to approach from the sides as well as from the region immediately upstream from the orifice.

Measurement of Pressure - Of major importance in tests on the enclosed jet is the pressure distribution in the upper corner of the sluice at the junction of the side and top transition curves. Therefore, piezometers were installed at close intervals in this region, proceeding downstream at increasing increments. For the sluice having sidewall revision No. 1 and roof revision No. 1, the piezometers were placed in the following manner proceeding downstream from the upper corner of the orifice plate: the first five piezometers were placed at 1/2 in. intervals, the next five were placed at 1 in. intervals, and the remaining two piezometers were 1-1/2 in. apart.

All of the piezometers were connected by rubber tubing to the manometer bank which was located just to the side of the orifice plate. The pressure measuring equipment, the process of determining the zero for the manometer tubes, and the method of priming the piezometers was the same as had been described in the section on equipment and procedure for testing in Chapter II.

In making a test, the headwater was set to a given elevation, the piezometers primed, and then the pressures were determined by direct measurement of the difference in elevation of the meniscus and the zero mark. This measurement was made with a scale graduated in prototype feet. All measurements were tabulated on standard laboratory data sheets and plotted as describe in the following pages of this report.

Measurement of Discharge - Water was supplied to the 1:40 scale sluice-inlet model by a lateral from the main 8-in. pipe line and pump. As in the case of the sectional model, the 6-in. orifice meter and the mercury manometer were used to measure the discharge. The testing procedure consisted of setting the headwater at the desired elevation, and determining the discharge through the sluiceway by the orifice meter. All of the data were recorded on standard laboratory data sheets.

B. DISCUSSION OF TEST RESULTS

The sluice-inlet model, built to a 1:40 scale and installed in the side of the steel tank, consisted primarily of a rectangular sharp-edged orifice scaled to the size of the inlet transition at the upstream end. A horizontal floor, representing the sluice floor, extended downstream from the bottom edge of the orifice to the beginning of the vertical curve in the sluice bottom (see Fig. 18). Upstream from the orifice a corbel-type of entrance lip provided approach conditions similar to those for the sectional model (see Figs. 18 and 19).

It was planned that the shape of the side walls of the inlet transition was to be determined from the shape of the free jet. The transition sidewalls were then to be constructed and added to the model and the free surface of the top of the jet measured. From these curves, the shape of the sluice and transition roof was to be determined. Finally, the roof was to be constructed, installed, and checked for cavitation pressures.

Many difficulties in following this program were encountered, however, and deviations from the original schedule were made in the hope that time and money could be saved. These deviations, although they seemed to be logical at the time, may appear now to be out of order.

Shape of Free Jet

The first orifice tested was 16-1/2 ft wide by 35 ft high and was cut in the steel plate which was 1/4-inch thick. The size was governed by the maximum width of inlet which could be installed in the prototype without interfering with adjacent sluices, and by the maximum allowable height of emergency gate which could be built economically.

For the first test, the model was operated at maximum headwater. As the jet issued from the orifice, the general shape was rectangular, but the corners were beveled (see Fig. 19)due to the pattern of the approaching streamlines combined with the effect of the 90° corners of the orifice. As the jet progressed downstream, the beveled corners became increasingly pronounced thereby explaining why the minimum pressures for a rectangular inlet transition are to be found in the upper corners (see Fig. 2). Downstream from the orifice plate, as may be seen in Fig. 19, the beveled corners quickly became wider until the entire cross section of the jet was altered and the shape of the jet had no resemblance to the rectangular orifice through which it had just passed.

The measurements of the jet shape were then made using the 1/16-in. point gage pushed through the holes in the guide block (Fig. 18). The rod was held in the holes by friction and was adjusted until the end of the rod was just touching the flowing jet. From these measurements the shape of sidewall revision No. 2 of the inlet transition was obtained.

Form and Size of Inlet Transition

While these data were being obtained and plotted, it was decided to construct and test the inlet transition with sidewall revision No. 1. The transition sidewall curves were ellipses having the equation

$$\frac{x^2}{576} + \frac{y^2}{5.0625} = 1$$

in which x is the distance in feet parallel, and y is the distance perpendicular, to the sluice center line with the origin 24 ft downstream from the face of the dam at a point 8.25 ft from the sluice center line. The surface profile was measured with these sidewalls in place.

As explained previously, sidewall revision No. 2 corresponded to the shape of the sides of the free jet as determined from the model tests (see Fig. 19). These sidewalls were installed and surface profile measurements made to determine a shape for the roof transition. The beveled corners, and other surface irregularities which existed in the free jet, also appeared with the sidewalls in place. Therefore profile measurements were made adjacent to the sidewalls and 1.75 ft and 3.3 ft out from the walls. Due to the beveled edges, the measurements at the sidewalls gave a profile somewhat lower than those nearer the center line of the jet. As may be seen in Fig. 20, the total drop in the water surface adjacent to the sidewalls at the point where the gate would be installed was approzimately 7 ft below the top edge of the orifice.

Next, an attempt was made to fit a roof curve to an opening 33.33 ft high at the face of the dam and 20.33 ft high at the gate. To this end, roof revision No. 1 was set up as

$$y^2 + \frac{x^2}{4} - 13x = 0$$

which fit the two controlling heights. This roof was constructed and installed with sidewall revision No. 2 and the resulting pressure distribution measured. As shown in Fig. 21, the minimum pressure, 19.8 ft below atmospheric, was only slightly greater than that in the original sluice design. Since no marked improvement in pressures was obtained, it was decided to make a further roof revision based more directly upon the shape of the free water surface.

As shown in Figs. 19 and 20, the top of the orifice during the measurements of the free water surface was 35 ft above the floor of the sluice. The 7 ft drop or contraction made the sluice height 28 ft which necessitated a gate considerably higher than was desired. This curve of the water surface appeared usable, however, as a trial roof curve for an orifice somewhat smaller in height. Therefore, it was decided to construct roof revision No. 2, using a curve having the general shape of the free water surface from the 35 ft orifice but with the entire surface lowered to provide a sluice and gate height of 20.33 ft. To have a more gradual transition than was indicated by the data, the height of the inlet entrance was increased to 28.66 ft and a curve arbitrarily drawn from this point to become tangent to the adjusted water surface curve further downstream. It was anticipated that such a curve would not exactly represent the curve for an orifice 28.66 ft high but it was believed that the difference would be minor.

The pressure distribution resulting from roof revision No. 2 and sidewall revision No. 2, shown in Fig. 22, gave a minimum value of 10 ft of water less than atmospheric. This was a considerable improvement over the first revision. However, it was believed that further improvement could be obtained by shaping the transition roof to the free jet surface curve of an orifice having a jet height of 20.33 ft at the gate. Preliminary to making these tests, however, this belief was confirmed by revising the existing model quickly with modeling clay to simulate the lower transition roof.

In view of the fact that the jet would not be reduced in height as much as the orifice, it was decided to make the orifice 25 ft high. Fig. 23 shows the results of these measurements. The beveled corners were still very pronounced and the drop in surface elevation adjacent to the sidewall at the gate was approximately 5.5 ft thereby making the sluice 19.5 ft high.

As a next step, it was decided to test roof transition No. 3 shown in Fig. 24. The curve fits the equation

$$19.51\, y^2 + x^2 - 50x = 0$$

and was used in an attempt to simplify the coordinate system necessary to describe the roof transition. Pressure measurements were made in the upper corner of this model using sidewall revision no. 2 and roof revision No. 3. The pressures dropped sharply about 3 ft inside the inlet to a minimum value of 9 ft of water less than atmospheric and held to this low value throughout the remainder of the distance to the gate. The minimum value was about the same as found for roof revision No. 2, but because the pressures were low over a greater length of the transition and because there was no material improvement over the second revision, roof revision No. 3 was considered undesirable.

For roof revision No. 4 the shape of the transition was based directly on the profile of the free jet issuing from the orifice 25 ft high and measured at the sidewall. With sidewall revision No. 2 and roof revision No. 4 in place, a minimum pressure of 10.4 ft of water less than atmospheric was obtained, as shown in Fig. 25. Although this low value was confined to a much shorter distance than with roof revision No. 3, it was totally unexpected and no definite explanation for it could be given. It is probable, however, that the curve as drawn from the data did not have a continually increasing radius of curvature from the inlet of the sluice to the gate. Had time permitted, the curve would have been differentiated graphically to determine what adjustments should be made.

It would have been desirable to continue the testing program to determine exactly the reasons for the low pressures in the sluice but the designer's time schedule did not permit further study. It is believed that further investigations would have revealed the cause of the low pressures and, with additional tests, an inlet having no pressures below atmospheric could have been developed. However, the shapes used in roof revision No. 4 were the most satisfactory of those tested and although the pressures in a relatively short length of the transition were considerably below atmospheric, the inlet was believed to be safe for prototype use for the following reasons: First, since only the upstream end of the sluice had been constructed in the model, the back pressure due to friction losses in the downstream portion of the prototype sluice was not included and second, a constriction, which was not included in the model, is necessary at the sluice outlet to force the sluice to flow full. Both influences will cause increased pressures in the prototype sluice.

The model inlet was calibrated and found to pass a quantity of water in excess of that which was actually needed. Consequently the downstream area of the sluice could be constricted to reduce the discharge and thereby raise the pressure gradient throughout the sluice. For the quantities involved and for the outlet dimensions chosen, this would amount to about 5 ft of head--the difference in velocity head between the sluice gate and the outlet. Losses from boundary resistance between the sluice gate and the exit would account for approximately another 4 ft of head.

In other words, if the entire sluice had been modeled--as it would have been if time and money had been available--the pressures in the model inlet would have been close to atmospheric. To demonstrate beyond a doubt, however, that downstream resistances would raise the sub-atmospheric pressures, the downstream end of the model inlet was constricted to raise the head on the piezometer nearest the gate by 7.5 ft, a value more conservative than those given above. The minimum pressure in the inlet was then found to be -4.2 ft of water, a figure which is believed to be safe. The pressure curve for this test, shown in Fig. 26 shows that small sub-atmospheric pressures are to be expected over only a short length of the inlet. For the remainder of the sluice the pressures will be well above atmospheric. Thus, the sluice roof revision No. 4 shown in Fig. 25 is considered to be safe for prototype use in combination with sidewall revision No. 2, Fig. 20, provided the sluice is pressurized as indicated above and shown in Fig. 26.

Chapter IV

AUXILIARY SPILLWAY MODEL

A. MODEL CONSTRUCTION AND TESTING

As planned during the design of the original model it was necessary to modify the service spillway model to study the operation of the auxiliary spillway. The problems involved in such a change are presented first, followed by a discussion of the model design and construction and the procedure and equipment for testing. Finally, the test results are examined and discussed.

Problems to be Considered

Since the spillways for the auxiliary and service sections are identical, the performance of the auxiliary spillway or the problems encountered need not be discussed here. The auxiliary sluice outlets, however, are somewhat different as shown in Fig. 27 and Plate 21. Therefore, it is necessary to study the flow emerging from the sluices, the flow pattern leaving the deflector bucket, and the erosion downstream from the bucket. Because the trajectory bucket is not a true energy dissipator and the energy will be dissipated downstream in the erodible portion of the streambed, it is necessary to check the erosion tendencies to ensure that the erosion which does occur will not undermine the structure.

Also to be considered is the effect on scour of (1) operating the overflow spillway with the sluices closed, (2) operating only the sluices with the overflow spillway gates closed, or (3) operating with alternate sluices and crest gates open.

The position and characteristics of the roller which forms below the bucket must be determined for all operating conditions. If the roller is too violent there is a possibility that eroded bed material will be swept into the bucket and cause damage to the prototype bucket through abrasive action.

Finally, the heights of the resulting waves and surges in the downstream channel should be measured for the different conditions of operation since they will cause bank damage unless adequate riprap or other protective devices are provided.

Design and Construction of Model

The design of the auxiliary spillway used in the model tests is shown in Fig. 27. Since the sluice design was entirely different from that used in the service spillway tests, it would have been necessary, in order to obtain complete similarity, to rebuild the sluices in the model. It was believed, however, that the small error in the sluice discharge resulting from retaining the original inlet would have little effect on tests to determine deflector bucket performance. Therefore, the original sluice inlets were retained and changes were made only in the outlet end. Because the design of the auxiliary spillway section provided for a pressure sluice, it was necessary to build the desired shape inside the existing sluices.

As pointed out in Chapter II, the roof of the downstream section was made of Honduras mahogany and fastened in place inside the sheet metal case. Thus, it was necessary only to remove this wooden insert and reform it to the desired shape of the sluices in the auxiliary spillway section (Fig. 27).

In order to reform the sluice floor and install the deflector bucket, wooden templets were made having the proper shape (see Fig. 27). The templets were cut so that the bottom conformed to the shape of the original model and the top to the shape of the sluice and bucket of the auxiliary spillway model. Finally, the templets were fastened in place and covered with 20-gage sheet metal to act as the surfacing material over which the water would flow.

Equipment and Procedure for Testing

The equipment used for testing the auxiliary spillway model was essentially the same as that for testing the service spillway model. No attempt was made to measure the pressures in the sluice inlets because these had been tested previously and a more satisfactory shape had been devised on the separate model. The trajectory curve of the sluice was much flatter than for the service spillway. For this reason and since the sluice was now pressurized, it was believed that there would be no problem of negative pressures on the sluice floor. The capacity of the sluices was assumed to be the same as for those in the service spillway.

In making the erosion tests, the river bed in the model was shaped to the correct level as shown in Plate 21. The model was then allowed to fill slowly with water, so as not to disturb the bed material, and the given conditions of head water and tail water were set.

Each test was run for a period of 20 minutes during which time sketches were made of the water surface and scour profiles as observed through the glass-walled section. Photographs were also made of the water surface and bed profiles after each run.

B. DISCUSSION OF TEST RESULTS

The service spillway discharges into a stilling basin which dissipates much of the energy in the flow before releasing it into the river. In the auxiliary section there is no stilling basin to dissipate energy, but rather, a curved bucket which projects the water downstream as far from the structure as possible. Because of the high rock elevation downstream from the auxiliary spillway it is not economically feasible to excavate a stilling basin.

With a trajectory bucket in operation, more severe conditions in the lower channel must be expected. Performance which would be considered unsatisfactory if it occurred for the stilling basin of the service spillway, may be the best operation obtainable with a trajectory bucket.

The bucket on the auxiliary spillway is not an energy dissipator. Only a negligible amount of energy is dissipated before the water is projected downstream. From a practical viewpoint, all of the energy is dissipated in the downstream channel and the jet leaving the bucket must erode a stilling pool downstream before optimum performance of the structure is obtained. It can be expected that the first prototype flows from the auxiliary spillway will cause extreme turbulence, high velocities, and large waves in the lower channel. Only after an energy dissipating pool has been eroded by the flow will optimum stilling action be obtained.

In any case, however, the dissipation of energy below the auxiliary spillway in a manner which results in a quiet water surface and very little erosion in the downstream channel cannot be achieved. On the other hand, the auxiliary spillway will operate infrequently and for only a few weeks at any one time. Also, for much of the time the river channel downstream from the auxiliary spillway will be relatively dry and therefore readily accessible--during which time the progress of river bed erosion in the prototype can be inspected and corrective measures taken.

The overfall crest of the auxiliary spillway is identical with that of the service spillway. As discussed in Chapter II, the service spillway crest was found to operate satisfactorily. Therefore, in this chapter consideration will be given only to a discussion of the test results on the trajectory bucket and the sluice discharge as it affected bucket operation.

Sluices

In rebuilding the service spillway apron for tests on the auxiliary spillway, it was necessary to rebuild the sluices to make the sluice flow enter the bucket in the proper direction. It was not necessary, however, to rebuild the sluices entirely for the study of the auxiliary section. It was believed that a small error in sluice discharge would have little effect on the tests to determine the performance characteristics of the energy dissipator. Consequently, the changes made in the sluices were near the outlet end--and were confined to those necessary to direct the water properly into the bucket, and those necessary to pressurize the sluice.

Since the half-sluice next to the glass panel was also reconstructed and the flow within the sluice could readily be seen, the flow pattern within the sluice is also described. It should be realized, however, that the flow conditions described and shown may be somewhat different from those which would have occurred had the recommended sluice been installed in the model. The changes made in the inlet transition curves could have modified considerably the flow conditions described at the gate.

With the head water at Elev. 630 the sluice was observed to flow almost entirely full. Immediately under the roof and downstream from the gate, however, a space approximately one foot high, containing a mixture of air and water was evident. This air apparently was entrained at the gate control section and was carried out through the sluice by the flowing water. The significance of the air pocket and its effect on flow conditions within the sluice was not determined.

Bucket

Of major concern in the study of the auxiliary spillway the action of the bucket in causing the water to be thrown downstream and away from the structure. Although only one bucket was tested, its effectiveness was judged not only from the actual results obtained in the model tests but also by evaluating the bucket performance on the basis of other buckets which have been tested for existing structures and with which the laboratory personnel was intimately familiar. Hydraulic model studies on the Apalachia, Ocoee No. 3, and Fontana Dam Spillways built by the Tennessee Valley Authority and the Hungry Horse, Davis, Anchor, and Keyhole Dam spillways under construction by the U. S. Bureau of Reclamation were made or observed by laboratory personnel and were used as a background in judging the performance of the bucket on the Hirakud auxiliary spillway. In addition, data were taken to indicate the type of operation to be expected in the prototype. In the following paragraphs the general performance of the structure, the wave heights and surges, and the erosion patterns, for various discharges and tailwater elevations are discussed.

General Performance - In auxiliary spillway operation, a great amount of energy must be dissipated in the lower channel. Considering this plus the fact that the auxiliary section will operate infrequently, the action and general performance of the bucket was considered acceptable. For headwater Elev. 630 and tailwater Elev. 538, the bucket directed the flow upward above the tailwater with the flow impinging again on the tailwater approximately 100 feet downstream from the end of the bucket (see Plate 22 and Fig. 28). This action created a reverse or ground roller which pulled bed material up against the bucket to protect it from undermining. Within the bucket itself, there was no tendency for a roller to form and little if any back water affect was caused by the overfall spillway discharge as it passed over the sluice outlets. With uniform gate and sluice operation no bed material entered the bucket.

Waves and surges in the lower channel were evident downstream from the plunging jet as was a deep erosion hole located well downstream from the bucket lip. Although the action was violent throughout the model the performance was considered acceptable and was probably as good as could be obtained, taking the previously-discussed conditions and limitations into account.

Waves and Surges - As may be seen in Fig. 28, the maximum waves downstream from the bucket occurred for headwater Elev. 630 and tailwater Elev. 525, and were 60 ft high. The wave heights were rather quickly reduced, and 400 ft downstream from the bucket were found to be 10 to 20 ft high. Waves of the latter magnitude existed throughout the lower end of the model test flume without appreciable reduction and consequently waves of this magnitude can be expected in the prototype structure.

When the tailwater was raised to Elev. 538 and Elev. 550, (Plates 22 and 23) the heights of the waves immediately downstream from the bucket were not materially reduced. In each of these cases the waves and surges were extremely large and fluctuation of considerable magnitude developed. With tailwater at Elev. 525 the water under the jet did not touch the undersurface of the jet immediately downstream from the lip of the bucket and an air pocket 20 ft high was intermittently formed and carried downstream. This action tended to deflect the jet periodically thereby increasing the wave action and the size of the scour hole.

When the headwater elevation was reduced to 590 (see Fig. 28 and Plate 24), the wave action was greatly reduced. In this case the maximum waves occurred for tailwater Elev. 525. At the bucket the waves were about 20 ft high and 300 ft downstream they were reduced to less than 10 ft in height (Plate 24). With tailwater Elev. 550 (see Plate 25), the waves were from 5 ft to 10 ft high at the bucket and from 2 to 5 ft high further downstream.

Tests were also conducted for headwater Elev. 630 with flow over the crest only, thereby reducing the discharge to approximately one-fourth of that which occurred with both the crest and sluices operating. Under these conditions higher waves were created immediately downstream from the lip of the bucket than were found with the sluices operating at headwater Elev. 590, probably because of the increased velocity (see Plate 26). Although these waves were in the neighborhood of 20 ft high, they soon reduced to less than 10 ft as they progressed downstream.

The waves in the channel are of importance in that they will cause bank damage unless adequate riprap or other protection is provided.

Erosion - The depth and extent of erosion depended on the quantity of water flowing, and the elevation of tailwater. An intermittent air pocket under the jet caused a sensitive balance of forces. Due to this situation, the size, shape, and location of the scour hole was constantly changing in the early stages of scour hole development. As may be seen in Fig. 28, the depth of scour varied considerably with the tailwater elevation. When the tailwater was at Elev. 550, the scour was to a depth of 30 ft, occurring approximately 160 ft downstream from the end of the bucket (Plate 23). When the tailwater was reduced to Elev. 538, the scour was to a depth of 40 ft (Plate 22), and finally when the tailwater elevation was at Elev. 525 the scour hole became 70 ft deep. Of significance is the fact that the scour hole, although very deep, was located approximately 230 ft downstream from the bucket so that it did not endanger the safety of the bucket or the structure in general. At lower discharges the erosion was less, as shown in Fig. 28 and Plate 24. The greatest depth of erosion with headwater Elev. 590 was 20 ft, and it occurred approximately 70 ft downstream from the end of the bucket lip (see Plate 24). This maximum erosion occurred for tailwater Elev. 525. When the tailwater was raised to Elev. 550 the scour depth was only 10 ft (Plate 25). It was apparent, therefore, that for low headwater elevations with only the sluices operating the flow and erosion patterns were satisfactory.

When the discharge was over the crest only (see Plate 26), the amount of scour was relatively negligible, being to a depth of about 10 ft.

Apparently then, the most severe operating conditions occur when the overfall spillway and the sluices are both operating with high headwater and low tailwater.

Because there is a possibility that one or more sluices may be out of operation during a flood, it is important to know the effect of such action in the stilling basin itself. Of particular importance is the action which results when several adjacent sluices are not operating. Although accurate predictions of transverse flow conditions could not be made in a sectional model, visual observations were made to determine the effect of one sluice, and two adjacent sluices, out of operation. With one sluice closed, three and one-half sluices open and no water spilling over the crest, a transverse circulation pattern was established in and below the bucket which caused very little additional erosion. With two or more adjacent sluices out of operation, however, the transverse circulation pattern carried bed material up into the bucket where, in the prototype, it could seriously damage the bucket through abrasive action. This same type of damage occurred in the bucket at Grand Coulee Dam as a result of necessary unsymmetrical operation during the construction period. Every precaution should be taken to avoid this type of operation in the prototype.

Further studies indicated that if the overfall spillway was discharging and two adjacent sluices were closed, the overfall spillway water swept any accumulation of material out of the bucket and thereafter prevented additional material from entering. In operating the structure, therefore, it is important that spillway gates be operated, if possible, as a substitute for sluices that may be out of operation.

Chapter V

RECOMMENDATIONS

Recommended Design

Overfall Spillway - The action of the overfall portion of the spillway was entirely satisfactory and the original design is recommended for this part of the structure (Dwg. 5 and Fig. 8). Although it might be possible to eliminate the fin downstream from each spillway pier where the nappes from adjacent bays meet, the benefits which would be derived from such an improvement would be so small in this case that corrective measures are not recommended.

Sluices - Because of the influence of the overfall spillway discharges on the sluice outlets, the pressure sluice is recommended. The inlet transition of the sluice which was developed from the model tests had the sides and top shaped as shown in sidewall revision No. 2 and roof revision No. 4 in Fig. 25. Tests on this inlet showed that pressures of about 4.2 ft of water below atmospheric existed in the inlet. This takes into account the necessary pressurizing of the sluices accomplished by constricting the outlets as shown in Fig. 26. Since the model studies were not run to conclusion, because of the necessity for meeting the time schedules of the designers, it is not known whether further adjustments in the inlet shape would have produced atmospheric pressures in the critical areas. In all probability, however, it would not be possible to gain more than a few feet of additional pressure under any circumstances, because of the limitations placed on the length of the inlet transition. The auxiliary spillway sluice shown on Dwg. 6 is therefore recommended for prototype use.

Although the pressures measured on the flow of the open channel sluices in the service spillway showed a low value of 14 ft of water below atmospheric at the point of curvature of the vertical curve, pressurizing the sluice will raise the pressure at least 6 ft, making the maximum subatmospheric pressure about minus 8 ft of water. However, it should be noted that the sluiceway shown on the final drawing, Dwg. 5, has been modified sufficiently to prevent making an accurate estimate of the pressures which will exist in the prototype. The equation of the sluice floor curve has been changed from:

$$y = 0.0044\,x^2 \quad \text{to} \quad y = 0.0021\,x^2$$

making a much flatter curve. To accomplish this change in design the beginning of the curve was moved upstream from near the gate seal to near the axis of the dam. The pressure values on the floor of the sluice probably will be raised by use of a flatter trajectory curve. In effect, moving the origin of the curve also tilts the inlet transition downward. The effect of the tilt on the inlet pressures is not known. In any case, it is believed that the changes made in the sluice design will result in improved overall performance.

It must be emphasized that the sluice for the service spillway, as shown on Dwg. 5, has not been tested. Predictions of its performance are based on comparisons with a sluice that is only generally similar. The deviations are not believed to be detrimental, and the sluice is therefore recommended for prototype use.

Stilling Basin for Service Spillway - The tests on the model of the stilling basin for the service spillway showed beyond a doubt that the apron should be constructed at Elev. 452 and that a dentated sill 20 ft high located at the end of the apron was necessary. The upstream face of the sill should be sloped at an angle of 30° with the horizontal and the dentils should have vertical upstream faces and occupy 50 percent of the sill face as shown in Fig. 11.

The necessary length of apron was not so conclusively determined, however, and a choice of apron lengths is possible. Although the apron which extended 420 ft downstream from the axis of the dam provided the best performance on the basis of lower wave heights, it is believed that the apron can be shortened. As the apron length is reduced the wave action progressively becomes more intense, although the erosion does not, and it becomes necessary to judge the apron length required for the prototype on the basis of the effect of the waves on the river banks, earth dam, and dikes. Since this is not possible in a sectional model, the final choice should be made after a complete model of the spillway has been constructed and tested. The apron shown in Dwg. 5 and extending 331 ft below the axis is certainly the shortest apron which could be used, while the apron shown in Fig. 11 and extending 400 ft from the axis is probably as long as necessary.

Bucket for Auxiliary Spillway - The trajectory bucket shown in Dwg. 6 will operate as intended but the action below the bucket can be expected to be extremely violent. Although large waves will be formed in the lower channel, erosive action close to the bucket will not be dangerous. With the understanding that flow in the lower channel will appear to be extremely violent, the trajectory bucket shown in Dwg. 6 is recommended for the prototype.

Additional Model Tests

In a project of this size and complexity it is desirable and it will be found to be profitable to continue the model testing until the possibilities for improvements in design and reductions in cost are exhausted. For the Hirakud project this will amount to a sizeable test program. Every phase of each hydraulic feature should be examined carefully by model tests to be certain that the structure operates as intended and that the cost of the structures be kept as low as possible. In this respect model tests will not be expensive, as their cost will be returned many times over, not only in actual construction savings but in added confidence in the completed structure. It is therefore recommended that model tests be continued and additional information obtained as outlined below.

Sluices - Models of the service and auxiliary section sluices should be constructed and tested to be certain that excessive subatmospheric pressures do not exist in the inlet or the barrel of the sluice. As explained in the report, lack of time and funds prevented completion of these studies and it is of utmost importance that the best possible sluice design be obtained. Any difficulty found in one prototype sluice will also be found in the other 83 sluices.

Complete River Model - The entire spillway section of the project should be modeled and, if possible, the river banks should be included within the model limits. Sufficient upstream and downstream topography should be modeled to provide proper approach and exit conditions and a portion of the earth embankment on the left side should be constructed.

Tests should be made using the apron and bucket recommended as a result of these tests and the effect of wave heights on the earth dam, river banks, and dike should be studied. Longer aprons should be tested to determine their effect on reducing the wave action and a bucket with a slightly flatter trajectory should be investigated for the same reason. It is known from past experience that the recommended bucket provided operation which was very nearly the best, if not the best, operation obtainable. It is possible, however, that a bucket of the same radius but with a slightly flatter trajectory might improve the performance. Because of the great amount of energy to be dissipated, the results could not be markedly improved. Other tests might be made on the trajectory bucket to determine whether ventilating the air pocket under the nappe leaving the bucket lip would improve the stability of the jet and result in a quieter water surface downstream. A dentated bucket lip might help to ventilate the pocket and improve the performance in other ways.

In all tests on the spillway, full consideration should be given to the size of the waves. In general there is a tendency to underestimate the damage that can be caused by waves of even medium height and large waves should be avoided if possible.

The training wall between the auxiliary and service spillways should be investigated to determine its necessary length and height. Erosion tendencies at the end of the wall should be studied for various combinations of open and closed gates and sluices in both the service and auxiliary spillways.

A schedule for operating the prototype structure should be carefully worked out since severe damage may be inflicted upon the trajectory bucket by improper manipulation of the sluices and the overfall spillway.

The effectiveness of the rock dike on the right in properly directing the flow down the river should be investigated as should the tendencies toward undermining or overtopping the dike for flood flows.

Other valuable information and data may be obtained from the model and it is possible that problems which cannot be foreseen at this time would also be evident. A complete model of the spillway is therefore an absolute necessity.

Navigation Lock and Powerhouse - The navigation lock should be model tested to a scale sufficiently large to determine the hydraulic characteristics of the filling and emptying systems. The behavior of ships in the chamber during filling should be carefully determined and the probable stresses in the mooring hawsers should be measured to be certain that filling and emptying of the lock will be accomplished in a smooth and uniform manner. Depending on where the lock chamber water is discharged while emptying the chamber, investigation of conditions at the outlet end of the emptying conduits should be made to determine the effect in the powerhouse tailrace and in the lower approach to the locks. Erosion below the emptying conduits may also be of concern.

APPENDIX

LIST OF DRAWINGS, FIGURES AND PLATES

DRAWINGS

Dwg. No.

1. Map of India (517-A-1)
2. Location Map (517-A-2)
3. Topography - General Plan & Profile (517-A-6)
4. Spillway Dam - Plan and Elevations (517-A-10)
5. Spillway Dam - Spillway Sections (517-A-11)
6. Spillway Dam - Sluices and Operating Gallery (517-A-22)
7. Spillway Dam - Service Spillway - Stilling Pool (517-A-26)
8. Area, Capacity and Discharge Curves (517-A-5)

FIGURES

Fig. No.

1. General Plan and Elevation
   1:40 Service Spillway Model

2. Sluice Layout, Piezometer Location, and Pressure Distribution
   Sluiceway - Original Design
   1:40 Service Spillway Model

3. Tailwater Rating Curve

4. Variation of Discharge with Head
   Overfall Spillway and Sluice - Original Design
   1:40 Service Spillway Model

5. Variation of Discharge and Discharge Coefficient with Head
   Overfall Spillway - Original Design
   1:40 Service and Auxiliary Spillway Model

6. Variation of Discharge and Discharge Coefficient with Head
   Sluiceway - Original Design
   1:40 Service Spillway Model

Fig. No.

7. Piezometer Location, Pressure Distribution, and Water Surface Profile
Overfall Spillway Crest - Original Design
1:40 Service and Auxiliary Spillway Model

8. Permanent Spillway Sections
Sloping Apron - Trajectory Bucket
Preliminary Drawing for Model Studies

9. Water Surface and Erosion Profiles
Effect of Sill Design
Apron El. 452 - Overall Length 420 ft
1:40 Service Spillway Model

10. Water Surface and Erosion Profiles
Effect of Sill Design
Apron El 452 - Overall Length 420 ft
1:40 Service Spillway Model

11. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 452 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

12. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 452 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

13. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 472 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

14. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 472 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

15. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 462 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

16. Water Surface and Erosion Profiles
Effect of Apron Length
Apron El 462 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

17. Piezometer Location and Pressure Distribution on Sill
Effect of Sill Location
Apron El 452 - 20 ft, 27° Dentated Sill
1:40 Service Spillway Model

Fig. No.

18. General Plan and Elevation
1:40 Sluice Inlet Model

19. Free Jet - Cross Sections
1:40 Sluice Inlet Model

20. Top Profile
Sidewall Revision No. 2 - Inlet 35 ft High
1:40 Sluice Inlet Model

21. Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 1
1:40 Sluice Inlet Model

22. Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 2
1:40 Sluice Inlet Model

23. Top Profile
Sidewall Revision No. 2 - Inlet 25 ft High
1:40 Sluice Inlet Model

24. Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 3
1:40 Sluice Inlet Model

25. Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 4
1:40 Sluice Inlet Model

26. Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 4
Pressurized Sluice
1:40 Sluice Inlet Model

27. Elevation
1:40 Auxiliary Spillway Model

28. Water Surface and Erosion Profiles
1:40 Auxiliary Spillway Model

PLATES

Plate No.

1. Glass-walled Channel - Sectional Model
River Bed Molded to Shape for Testing
1:40 Auxiliary Spillway Model

Plate No.

2. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 525, 20-min Erosion Test

3. Head Box - Sectional Model
Transition Framework Ready for Plywood Covering

4. Head Box - Sectional Model
Partially Covered Transition Framework

5. Head Box - Sectional Model
General View Showing Whalers, Hook Gage Well, and Spillway Structure

6. Head Box - Sectional Model
Partially Covered Transition Framework and Upstream Face of Dam

7. Spillway Structure - Sectional Model
Completed Assembly of Model

8. Spillway Structure - Sectional Model
Completed Concrete Base and Crest
Framework for Sluice Assembly

9. Spillway Structure - Sectional Model
Completed Concrete Base, Spillway Framework, and Crest Assembly

10. Spillway Structure - Sectional Model
Crest Assembly Showing Templets, Piezometer Section, and Guide Tubes for Sluice Gate Control Arms

11. Spillway Structure - Sectional Model
Completed Crest Showing Pier Anchor Bolts, Piezometer Section, and Guide Tubes for Sluice Gate Control Arms

12. Water Surface and Erosion Profiles - Service Spillway
40 ft High Trapezoidal Sill 420 ft from Axis of Dam
HW 625, TW 520, Apron 452

13. Water Surface and Erosion Profiles - Service Spillway
40 ft, 34° Dentated Sill 420 ft from Axis of Dam
HW 630, TW 520, Apron 452

14. Water Surface and Erosion Profiles - Service Spillway
10 ft, Non-dentated Sill 420 ft from Axis of Dam
HW 630, TW 520, Apron 452

15. Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 400 ft from Axis of Dam
HW 630, TW 520, Apron 452

16. Water Surface and Erosion Profiles - Service Spillway
20 ft, 27° Dentated Sill 400 ft from Axis of Dam
HW 630, TW 550, Apron 452

Plate No.

17. Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 300 ft from Axis of Dam
HW 630, TW 520, Apron 452

18. Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 336.7 ft from Axis of Dam
HW 630, TW 520, Apron 452

19. Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 360 ft from Axis of Dam
HW 630, TW 525, Apron 472

20. Sluice Inlet Model
General View Showing Water-supply Piping, Entrance Tank, Orifice Plate, Sluice Floor, Hook Gage Well, and Manometer Board

21. Glass-walled Channel - Sectional Model
Spillway Structure and River Bed Prepared for Testing

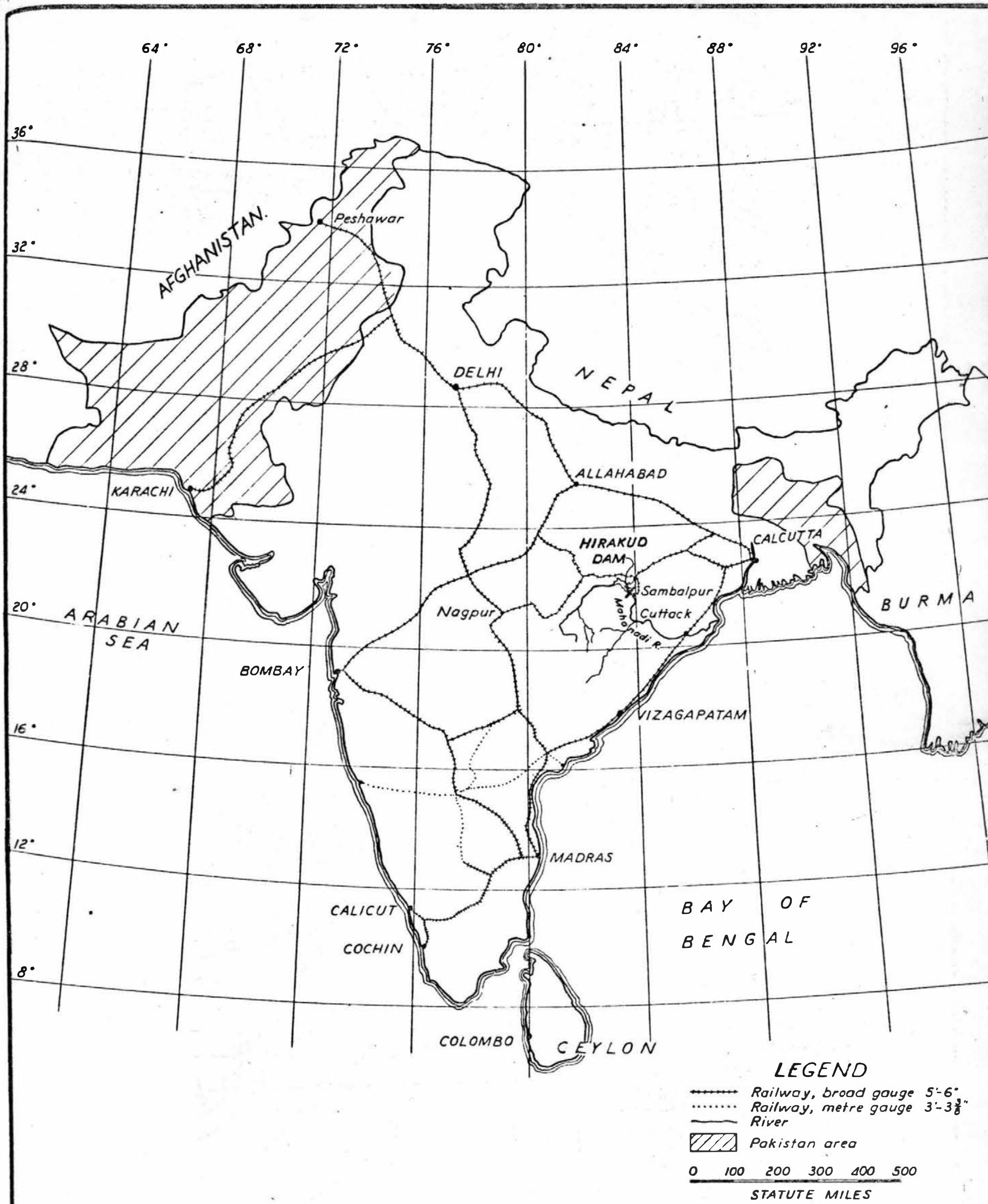
22. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 538, 20-min Erosion Test

23. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 550, 20-min Erosion Test

24. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 590, TW 525, 20-min Erosion Test

25. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 590, TW 550, 30-min Erosion Test

26. Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 525, Crest Flow Only, 20-min Erosion Test



GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION

HIRAKUD DAM

MAP OF INDIA

APPROVED J. L. Savage
CONSULTING ENGINEER

CWINC — INDIA

REVIEWED
S C Desai
APPROVED

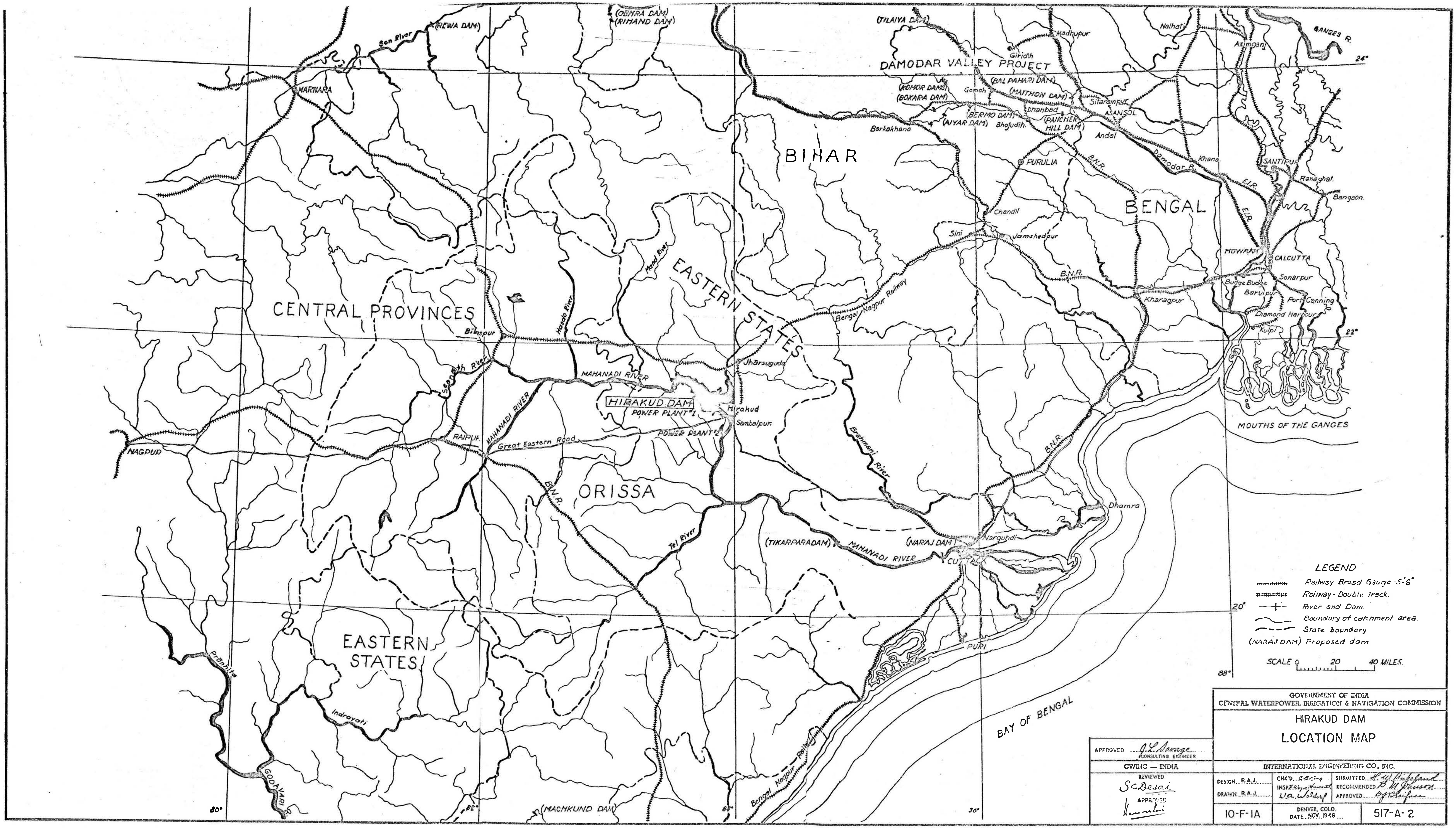
INTERNATIONAL ENGINEERING CO., INC.

DESIGN A.A.F.
DRAWN A.A.F.
CHK'D. C. R. King
INSP.
SUBMITTED
RECOMMENDED B. M. Johnson
APPROVED

10-F-1

DENVER, COLO.
DATE NOV. 1949

517-A-1


DAMODAR VALLEY PROJECT
BIHAR
BENGAL
CENTRAL PROVINCES
EASTERN STATES
ORISSA
EASTERN STATES
HIRAKUD DAM
POWER PLANT #1
POWER PLANT #2
Hirakud
Sambalpur
Jharsuguda
MAHANADI RIVER
Great Eastern Road
NAGPUR
RAIPUR
Bilaspur
Seonath River
Hasdo River
Mand River
Bengal Nagpur Railway
Brahmani River
Tel River
Indravati
Pranhita
GODAVARI R.
(TIKARPARA DAM)
(NARAJ DAM)
Narguhdi
CUTTACK
PURI
Dhamra
CALCUTTA
HOWRAH
Kharagpur
Jamshedpur
Sini
Chandil
PURULIA
ASANSOL
Sitarampur
Andal
Dhanbad
Gomoh
Bhojudih
Barkakhana
Giridih
Madhupur
Nalhati
Azimganj
GANGES R.
SANTIPUR
Ranaghat
Bangaon
Khana
Damodar R.
Sonarpur
Budge Budge
Baruipur
Port Canning
Diamond Harbour
Kulpi
MOUTHS OF THE GANGES
BAY OF BENGAL
Son River
MARWARA
(REWA DAM)
(OBHRA DAM)
(RIHAND DAM)
(TILAIYA DAM)
(BAL PAHARI DAM)
(KONAR DAM)
(BOKARA DAM)
(MAITHON DAM)
(BERMO DAM)
(AIYAR DAM)
(PANCHET HILL DAM)
(MACHKUND DAM)
B.N.R.
E.I.R.
24°
22°
20°
80°
82°
84°
86°
88°
LEGEND
Railway Broad Gauge - 5'-6"
Railway - Double Track.
River and Dam.
Boundary of catchment area.
State boundary
(NARAJ DAM) Proposed dam
SCALE 0 20 40 MILES.
GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION
HIRAKUD DAM
LOCATION MAP
APPROVED J. L. Savage CONSULTING ENGINEER
CWINC — INDIA
REVIEWED S C Desai
APPROVED
INTERNATIONAL ENGINEERING CO., INC.
DESIGN R.A.J.
DRAWN R.A.J.
CHK'D
INSP'D
SUBMITTED
RECOMMENDED
APPROVED
10-F-1A
DENVER, COLO. DATE NOV. 1949
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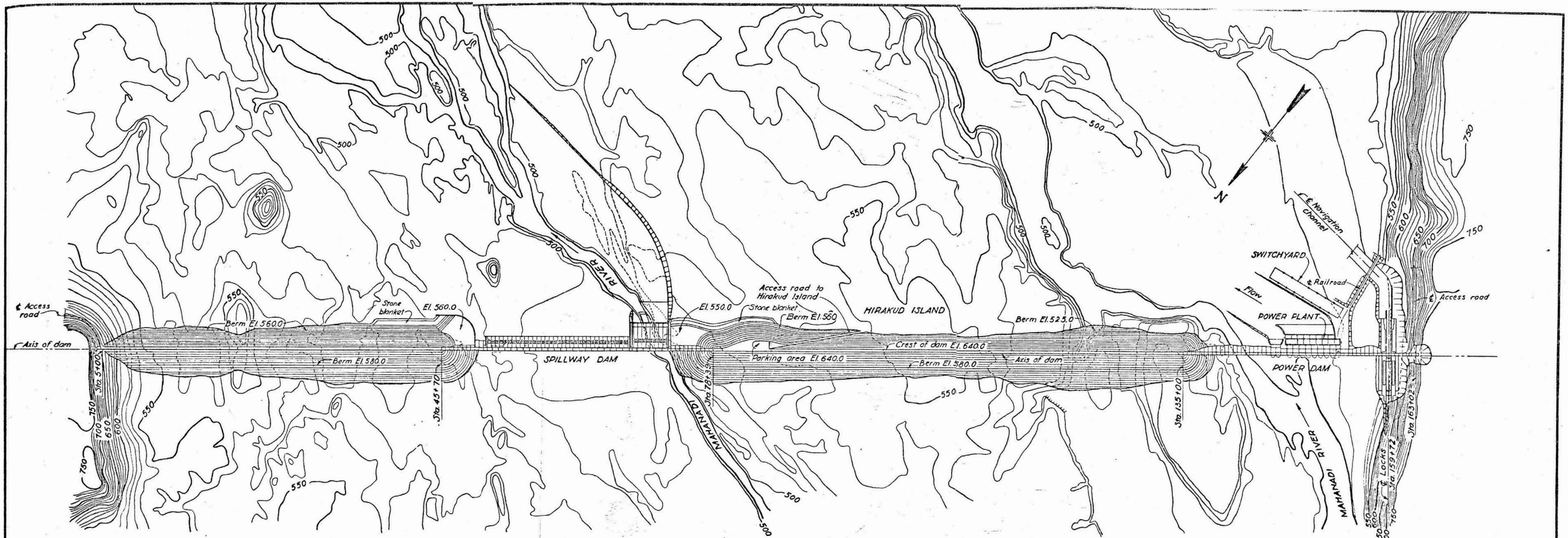
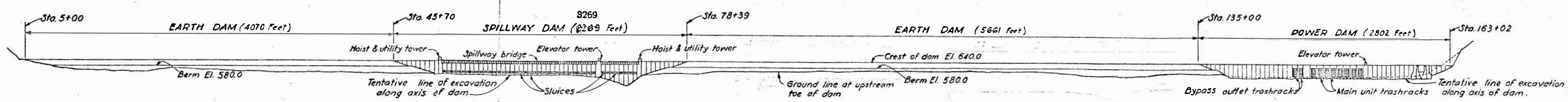


PLAN



UPSTREAM ELEVATION

SCALE 0 500 1000 FEET

NOTE

Tentative foundation excavation is based on an inspection note on drill cores Dec. 1949, by Dr. J.L. Savage. Actual required excavation will be determined in the field.

APPROVED J.L. Savage CONSULTING ENGINEER

CWINC — INDIA

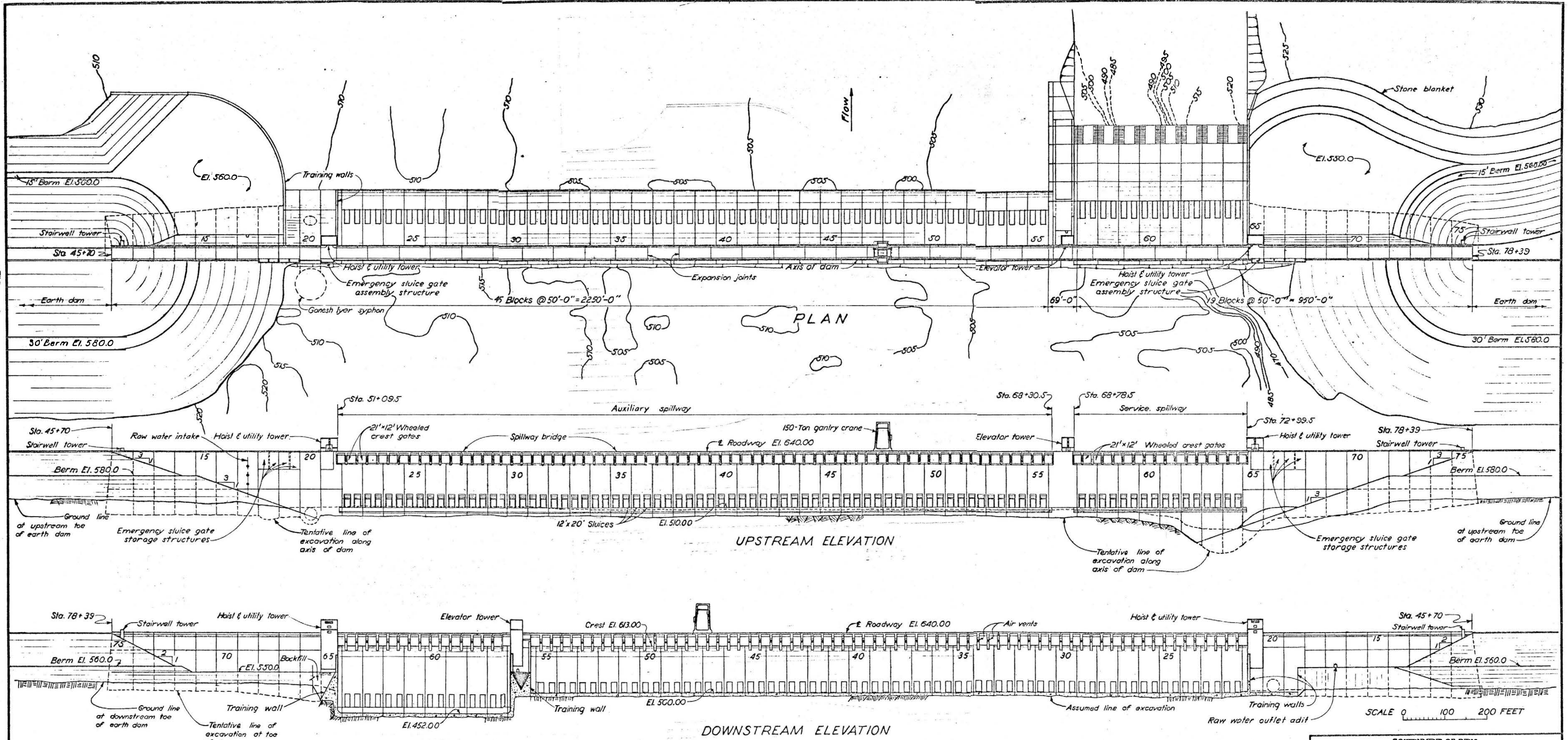
REVIEWED Sc Desai APPROVED

GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION

HIRAKUD DAM
TOPOGRAPHY
GENERAL PLAN & PROFILE

INTERNATIONAL ENGINEERING CO., INC.

DESIGN L.H.F.	CHK'D	SUBMITTED H.W. Birkeland
DRAWN L.H.F.	INSP.	RECOMMENDED
	W.A. Waldorf	APPROVED
10-F-4	DENVER, COLO. DATE NOV. 1949	517-A-6



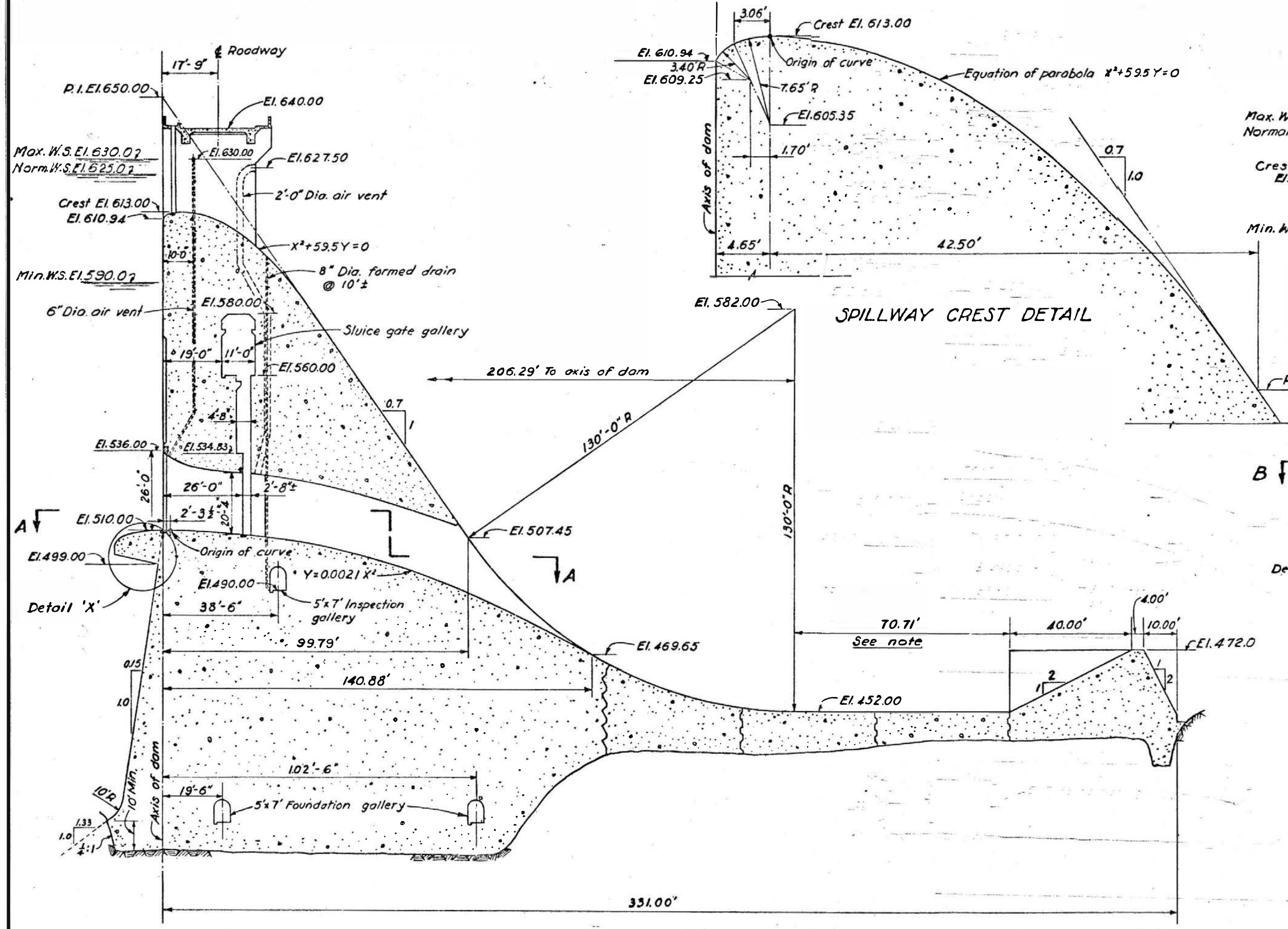
NOTES

All transverse contraction joints shall be keyed and grouted.

Crane capacity is given in tons of 2000 lbs.

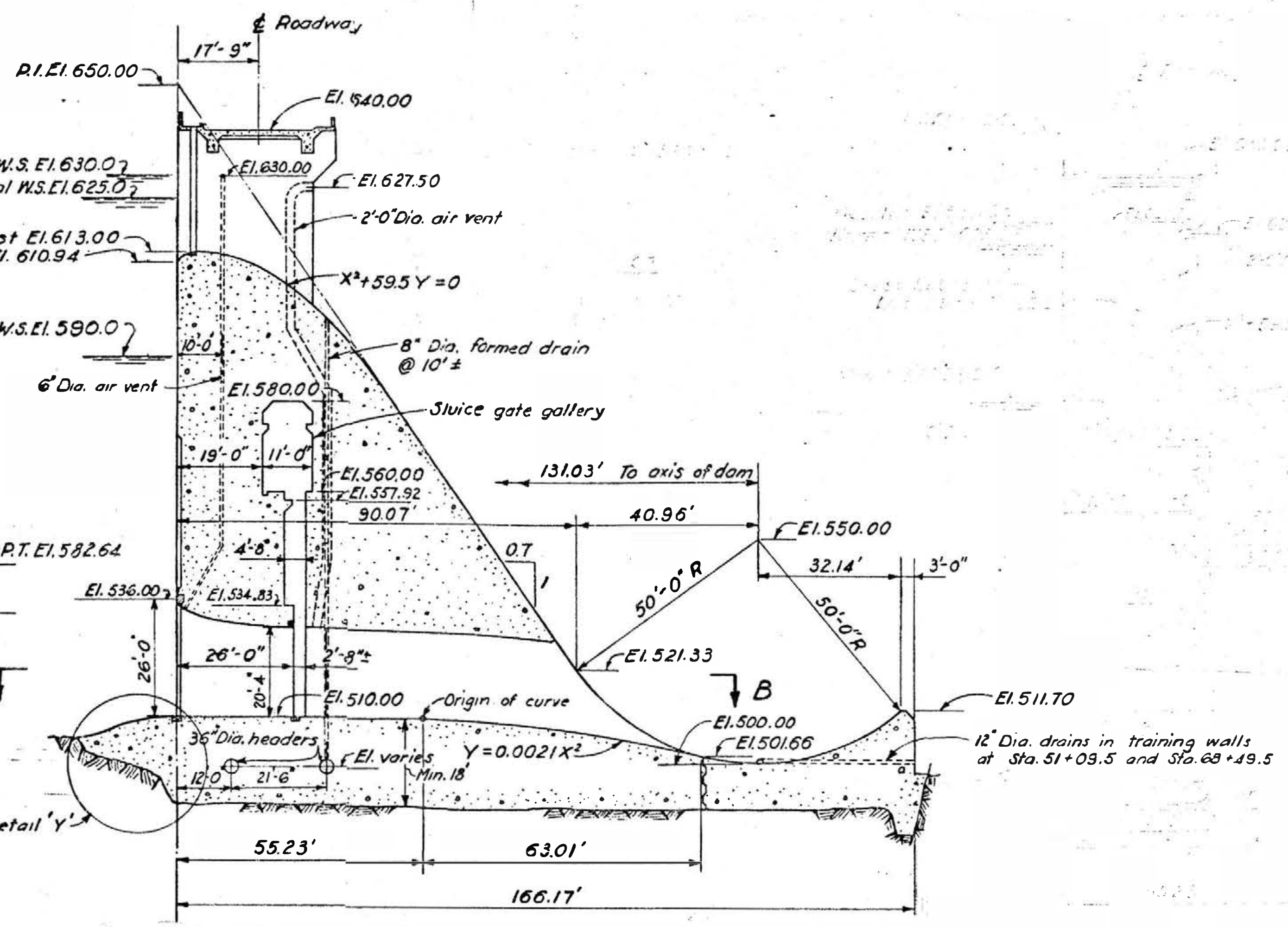
Tentative foundation excavation is based on inspection note on drill cores, Dec. 1948, by Dr. J. L. Savage. Actual required excavation will be determined in the field.

APPROVED J. L. Savage, CONSULTING ENGINEER	GOVERNMENT OF INDIA CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION
CWINC — INDIA REVIEWED Shantibhai Desai APPROVED [signature]	HIRAKUD DAM SPILLWAY DAM PLAN & ELEVATIONS
	INTERNATIONAL ENGINEERING CO., INC.
	DESIGN H.W.H. / DRAWN L.J.K. / CHK'D [signature] / INSP. [signature] / [signature] / SUBMITTED [signature] / RECOMMENDED [signature] / APPROVED [signature]
	11-F-1 / DENVER, COLO. DATE NOV. 1949 / 517-A-10

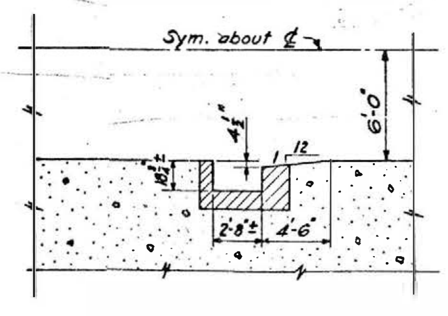


MAXIMUM SPILLWAY SECTION
SERVICE SPILLWAY

SPILLWAY CREST DETAIL



SHALLOW CHANNEL SPILLWAY SECTION
AUXILIARY SPILLWAY



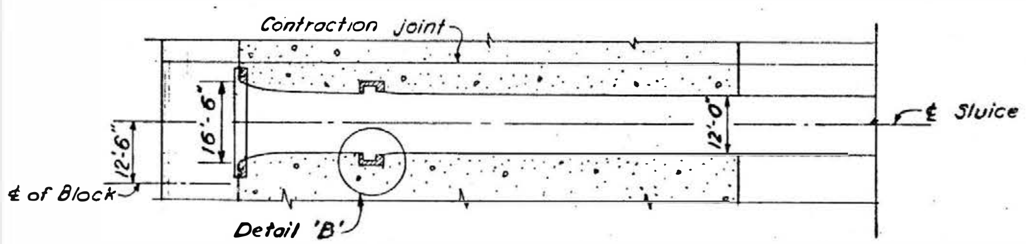
DETAIL 'B'

NOTES

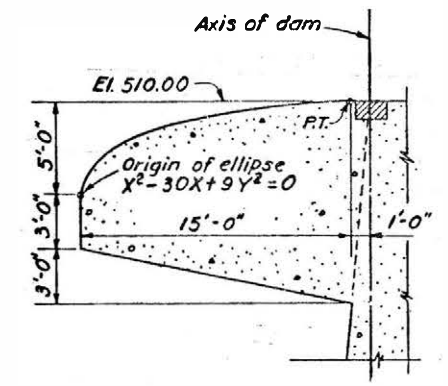
Strip overburden from rock surface for a distance of approximately 50 feet upstream from the axis of dam.
Tentative foundation excavation is based on an inspection note on drill cores, Dec. 1948, by Dr. J.L. Savage. Actual required excavation will be determined in the field.
For detailed dimensions of sluices see Dwg. 517-A-22.
Use detail "Y" except when stripped rock surface is below Elev. 505.0.
Length of stilling pool as shown is a minimum, based on limited hydraulic tests. The length may be changed when further tests have been completed.

SCALE 0 5 10 FEET
Details

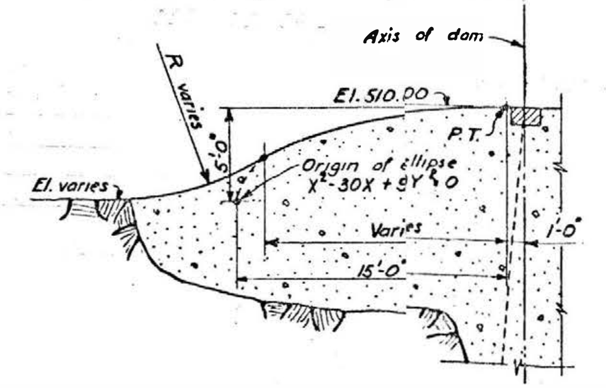
SCALE 0 20 40 FEET
Sections



SECTION A-A
SECTION B-B SIMILAR

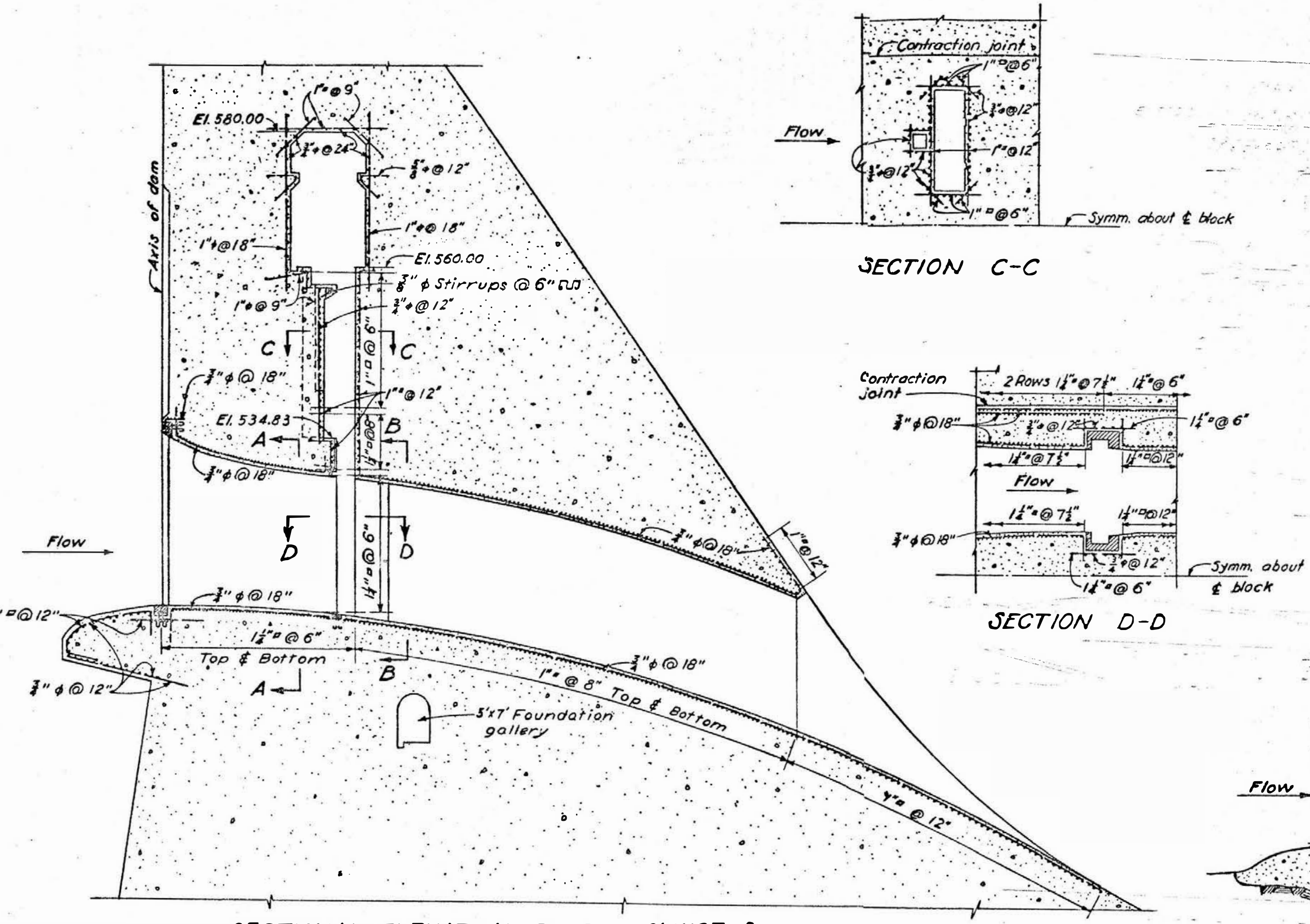


DETAIL 'X'



DETAIL 'Y'

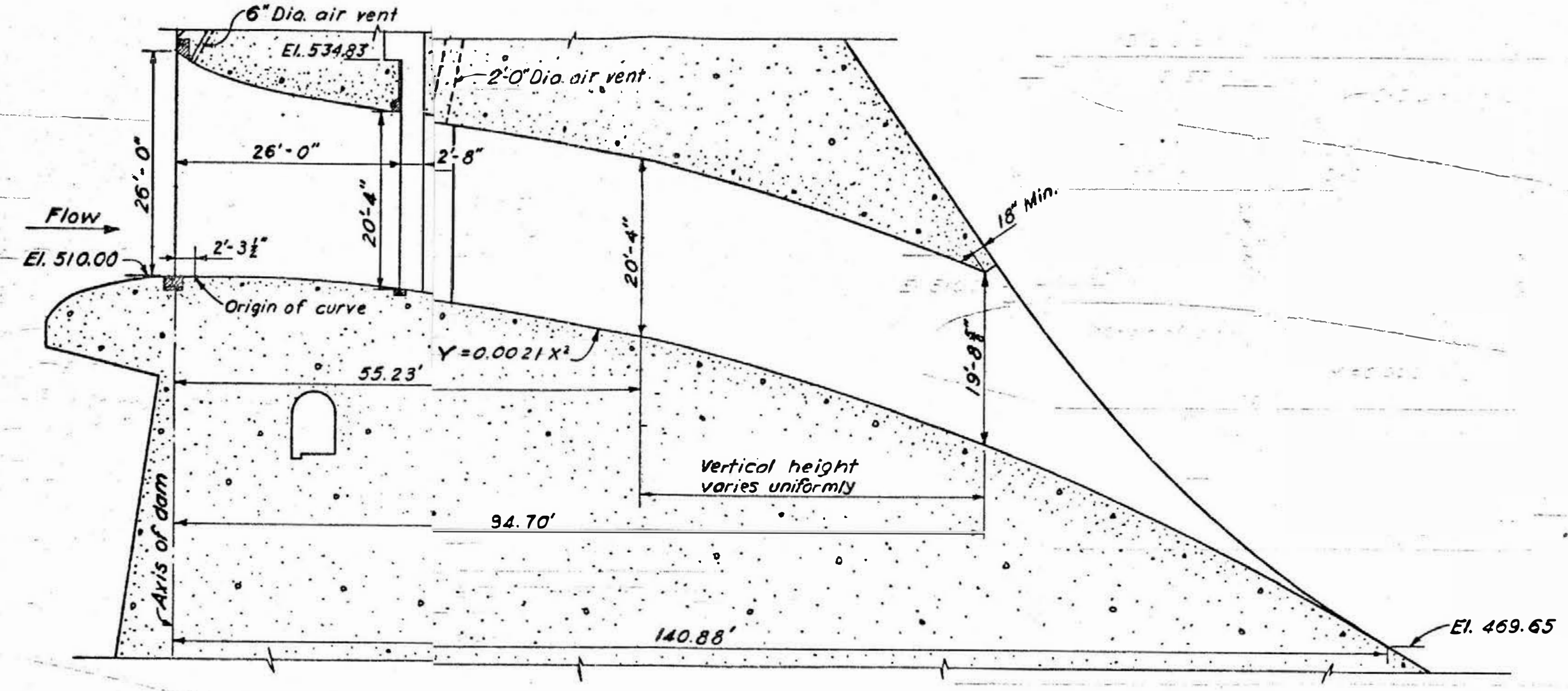
APPROVED J.L. Savage, CONSULTING ENGINEER	GOVERNMENT OF INDIA CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION
	HIRAKUD DAM SPILLWAY DAM SPILLWAY SECTIONS
CWINC — INDIA	INTERNATIONAL ENGINEERING CO., INC.
REVIEWED Shanti Desai APPROVED	DESIGN P.F.S.-W.A.E. · DRAWN L.H.F. · CHK'D · INSP. A.F. Samuel · SUBMITTED · RECOMMENDED · APPROVED
	11-F-3 · DENVER, COLO. DATE NOV. 1949 · 517-A-11



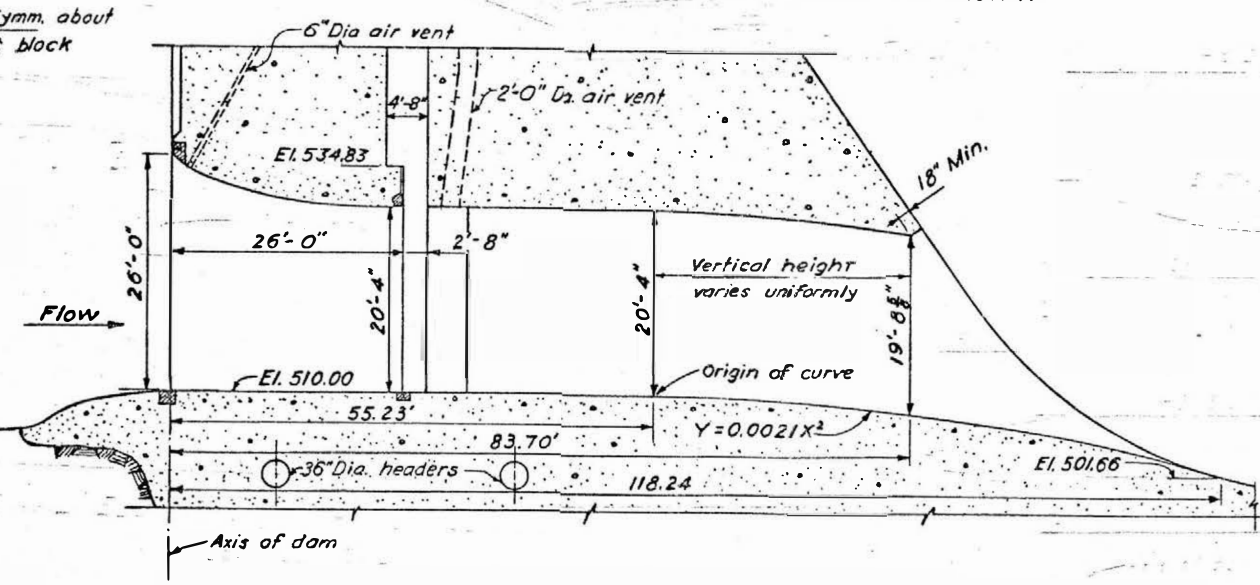
SECTIONAL ELEVATION THRU SLUICE & GALLERY
SERVICE SPILLWAY

SECTION C-C

SECTION D-D



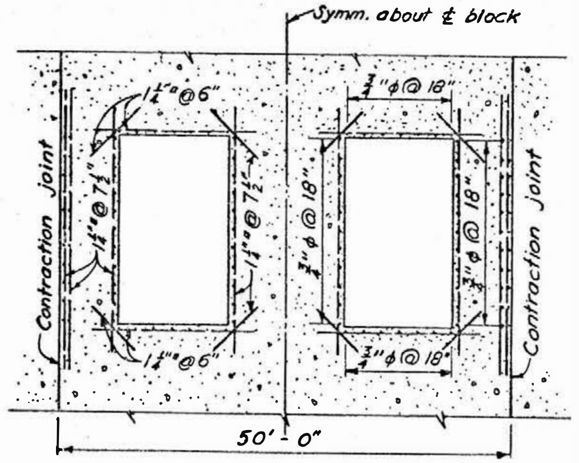
PROFILE OF SLUICE IN SERVICE SPILLWAY
(REINFORCEMENT NOT SHOWN)



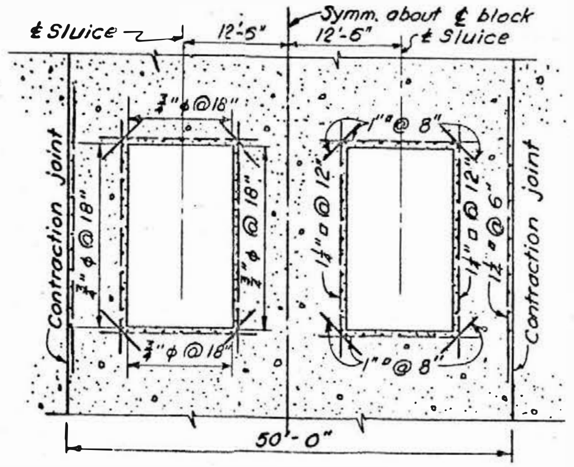
PROFILE OF SLUCE IN AUXILIARY SPILLWAY
(REINFOCEMENT NOT SHOWN)

NOTES

Reinforcement in sluice and gate gallery of auxiliary spillway is similar to that of service spillway.



SECTION A-A



SECTION B-B

BELLMOUTH COORDINATE TABLE

DISTANCE FROM AXIS	HEIGTH	WDTH
2"	25'-6 3/8"	14'-2"
4"	25'-4 1/2"	14'8 1/8"
6"	25'-2 3/4"	14'-4 1/8"
12"	24'-11 1/8"	13'-8 3/4"
18"	24'-8"	13'-4 1/4"
2'-0"	24'-5 1/8"	13'-0 7/8"
3'-0"	24'-0"	12'-7 3/8"
4'-0"	23'-7 3/8"	12'-3 1/8"
5'-0"	23'-3 1/8"	12'-0 7/8"
6'-0"	22'-11 1/2"	12'-0"
10'-0"	21'-11 1/8"	
14'-0"	21'-2 1/8"	
18'-0"	20'-8 1/8"	
22'-0"	20'-4 7/8"	
26'-0"	20'-4"	

SCALE 0 10 20 FEET

APPROVED J. L. Savage CONSULTING ENGINEER

CWINC — INDIA

REVIEWED Shantibhai Desai

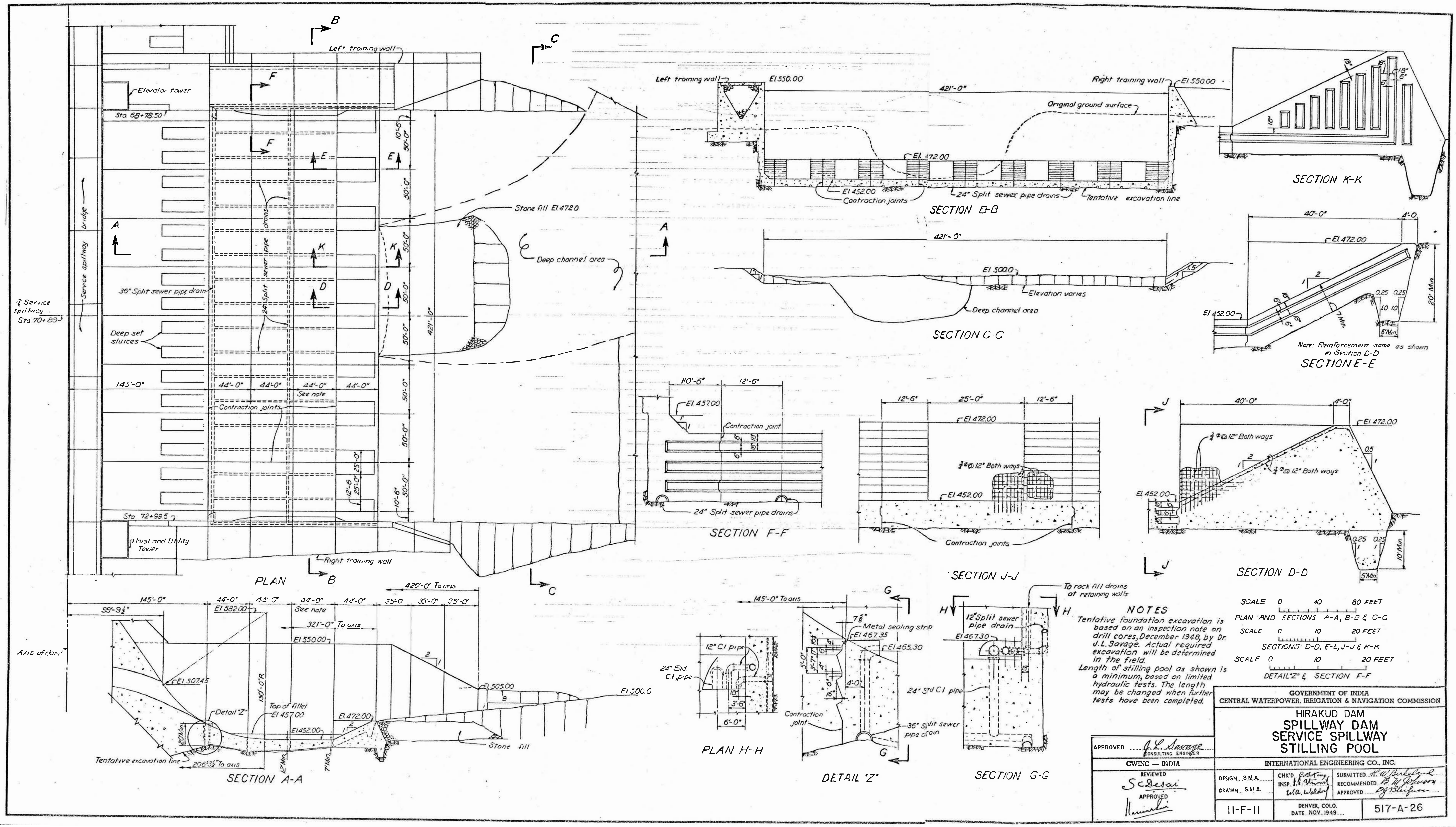
APPROVED [signature]

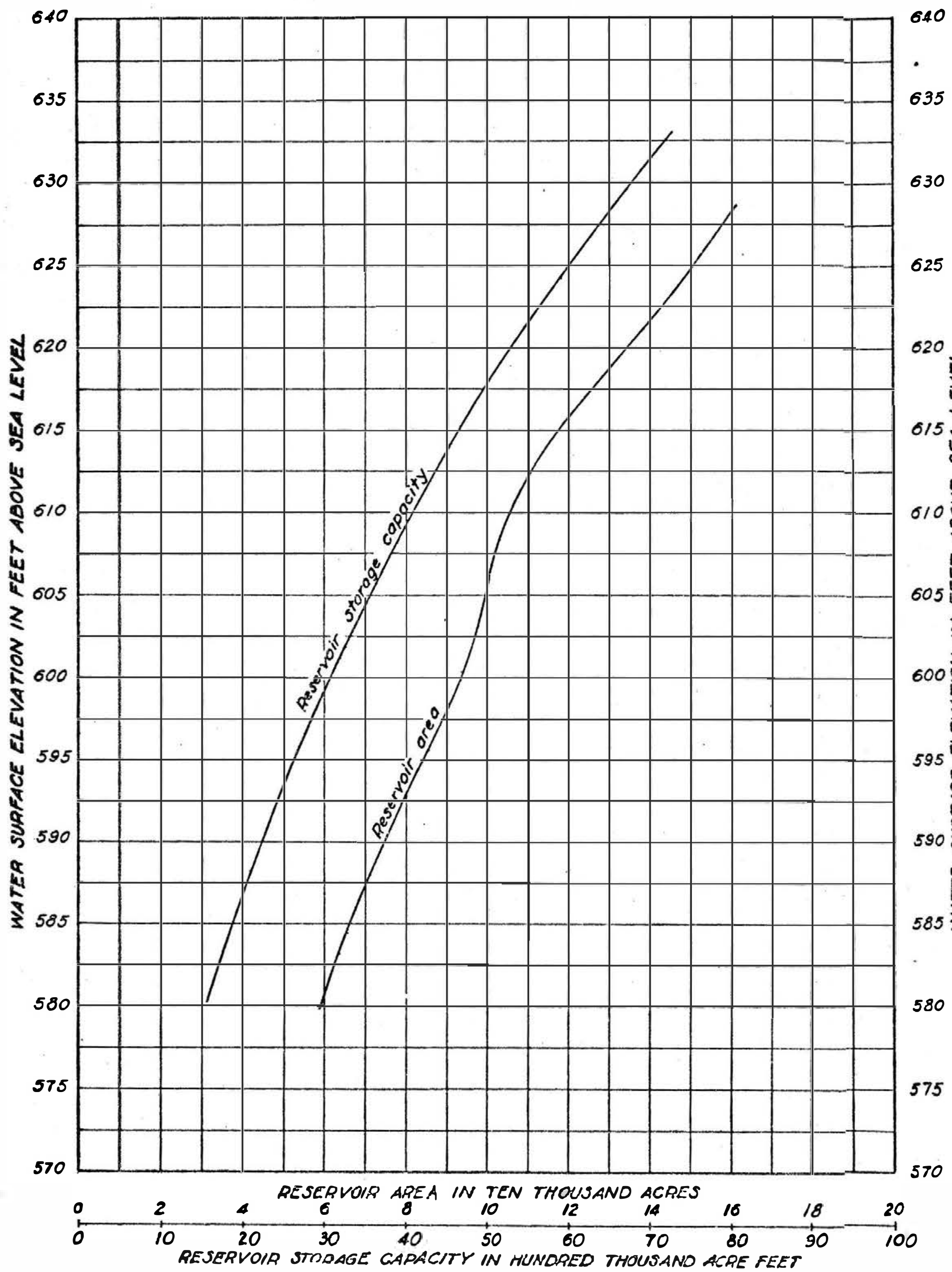
GOVERNMENT OF INDIA
CENTRAL WATERPOWER IRRIGATION & NAVIGATION COMMISSION

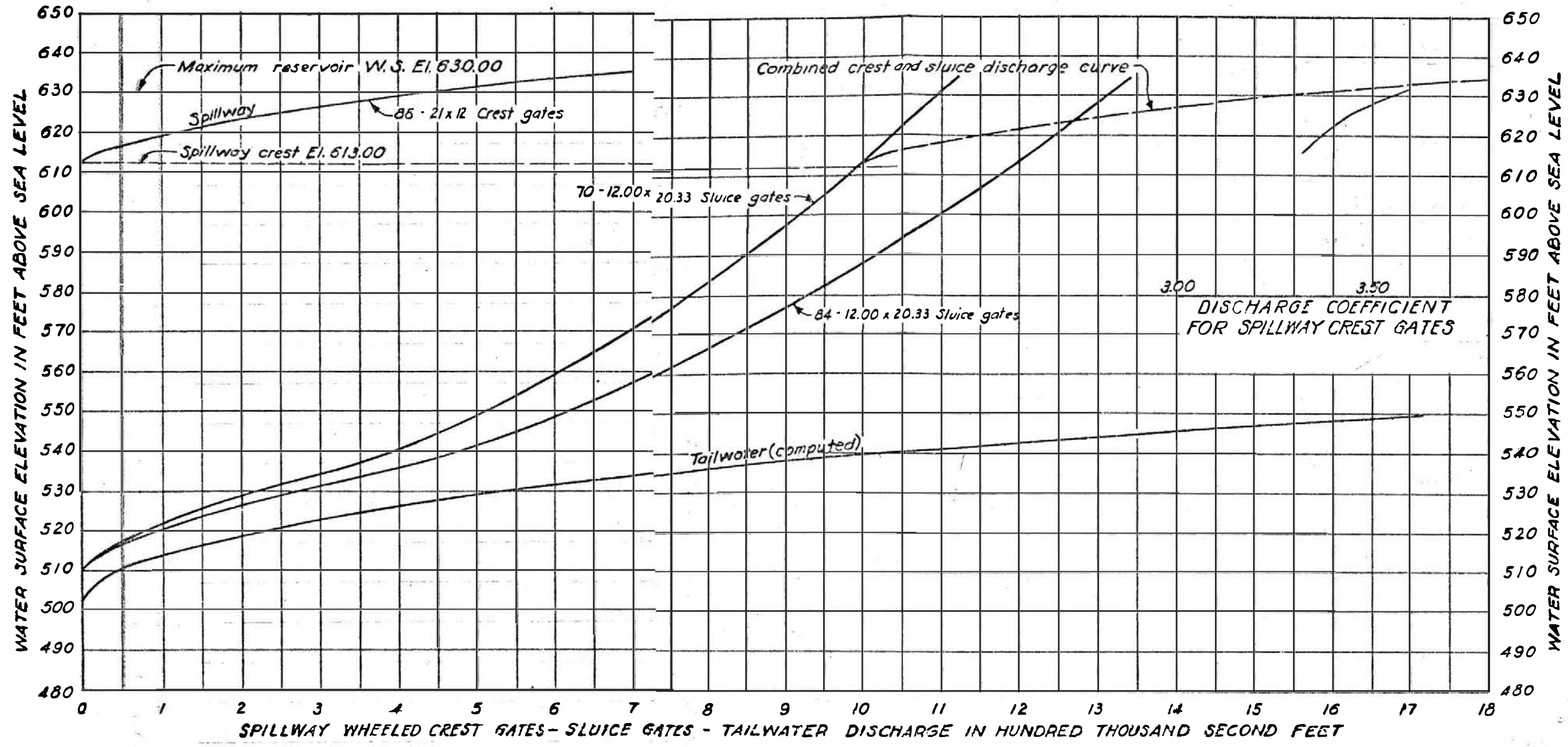
HIRAKUD DAM
SPILLWAY DAM
SLUICES & OPERATING GALLERY

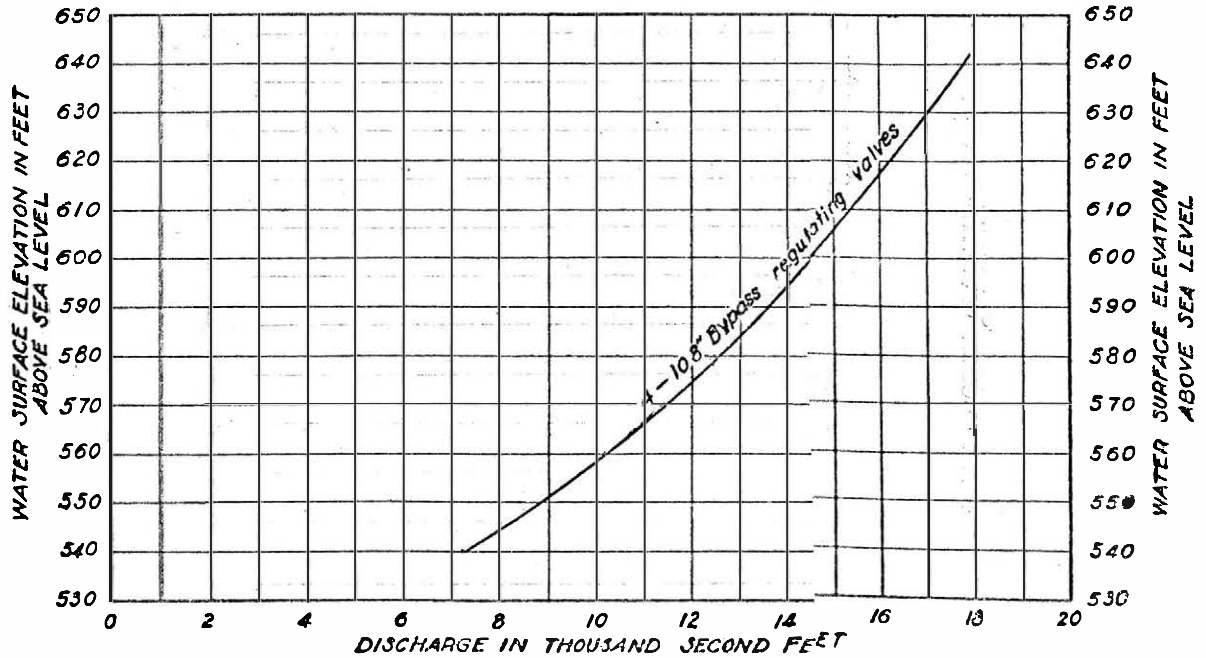
INTERNATIONAL ENGINEERING CO., INC.

DESIGN P.F.S. DRAWN L.H.F.	CHK'D [signature] INSP. [signature] W.A. Waldorf	SUBMITTED H. W. Birkeland RECOMMENDED B. H. Johnson APPROVED [signature]
11-F-14	DENVER, COLO. DATE NOV. 1949	517-A-22


PLAN
Left training wall
Right training wall
Elevator tower
Hoist and Utility Tower
Sta. 68+78.50
Sta. 72+99.5
Service spillway bridge
℄ Service spillway Sta. 70+89.3
36" Split sewer pipe drain
24" Split sewer pipe drains
Deep set sluices
Contraction joints
See note
Stone fill El. 472.0
Deep channel area
145'-0"
44'-0"
50'-0"
421'-0"
SECTION B-B
Left training wall
El. 550.00
421'-0"
Right training wall
Original ground surface
El. 472.00
El. 452.00
Contraction joints
24" Split sewer pipe drains
Tentative excavation line
SECTION C-C
El. 500.0
Elevation varies
Deep channel area
SECTION K-K
SECTION E-E
Note: Reinforcement same as shown in Section D-D
SECTION F-F
El. 457.00
Contraction joint
24" Split sewer pipe drains
SECTION J-J
El. 472.00
El. 452.00
3/4"⌀ @ 12" Both ways
Contraction joints
SECTION D-D
El. 472.00
El. 452.00
3/4"⌀ @ 12" Both ways
SECTION A-A
426'-0" To axis
321'-0" To axis
99'-9 1/2"
Axis of dam
El. 582.00
El. 550.00
El. 507.45
El. 505.00
El. 500.0
Top of fillet El. 457.00
El. 452.00
El. 472.00
Detail "Z"
Stone fill
Tentative excavation line
206'-3 3/8" To axis
PLAN H-H
12" C.I. pipe
24" Std. C.I. pipe
DETAIL "Z"
145'-0" To axis
Metal sealing strip
El. 467.35
El. 465.30
Contraction joint
SECTION G-G
To rock fill drains of retaining walls
12" Split sewer pipe drain
El. 467.30
24" Std. C.I. pipe
36" Split sewer pipe drain
NOTES
Tentative foundation excavation is based on an inspection note on drill cores, December 1948, by Dr. J.L. Savage. Actual required excavation will be determined in the field.
Length of stilling pool as shown is a minimum, based on limited hydraulic tests. The length may be changed when further tests have been completed.
SCALE 0 40 80 FEET
PLAN AND SECTIONS A-A, B-B & C-C
SCALE 0 10 20 FEET
SECTIONS D-D, E-E, J-J & K-K
SCALE 0 10 20 FEET
DETAIL "Z" & SECTION F-F
GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION
HIRAKUD DAM
SPILLWAY DAM
SERVICE SPILLWAY
STILLING POOL
INTERNATIONAL ENGINEERING CO., INC.
APPROVED J.L. Savage CONSULTING ENGINEER
CWINC — INDIA
REVIEWED S.C. Desai
APPROVED
DESIGN S.M.A.
DRAWN S.M.A.
CHK'D
INSP.
SUBMITTED
RECOMMENDED
APPROVED
11-F-11
DENVER, COLO. DATE NOV. 1949
517-A-26







NOTES

Tailwater curve shown is modified from tailwater curve at Hirakud (Sambalpur) Gage [Hirakud Dam Project - June 1947]. Modification is for single channel flow to left of Hirakud Island.

Sluice discharge computed using formula $Q = 0.80 A\sqrt{2gh}$.

Crest discharge coefficient determined by model and modified to include friction losses not represented by the model. The crest discharge was computed from the formula $Q = CLH^{\frac{3}{2}}$.

Bypass discharge computed from the formula $Q = 0.8 A\sqrt{2gh}$

APPROVED J. L. Savage
CONSULTING ENGINEER

CWINC — INDIA

REVIEWED S C Desai

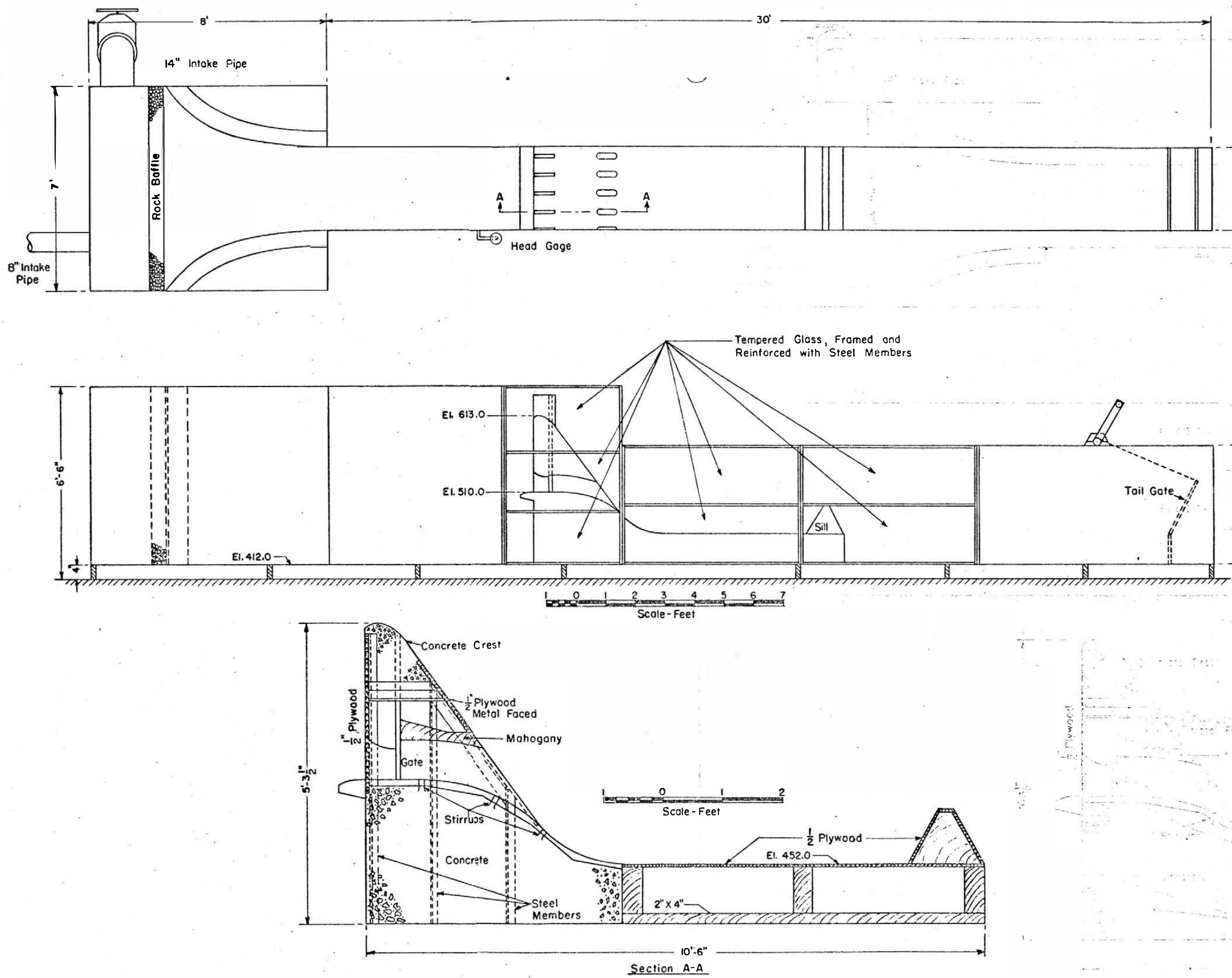
APPROVED [signature]

GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION & NAVIGATION COMMISSION

HIRAKUD DAM
AREA, CAPACITY & DISCHARGE CURVES

INTERNATIONAL ENGINEERING CO., INC.

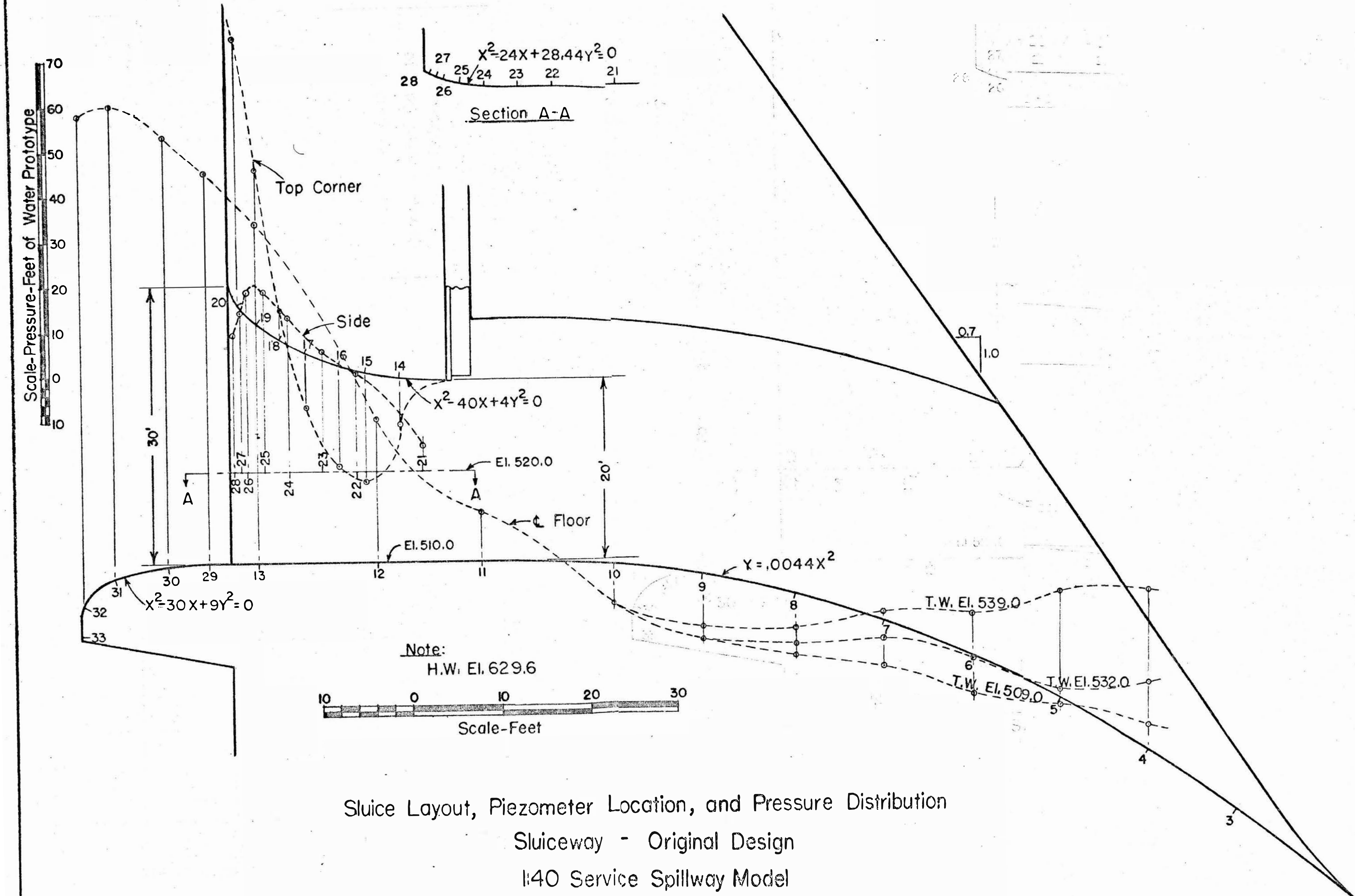
DESIGN R.A.J.-L.H.F.	CHK'D [signature]	SUBMITTED H. W. Birkeland
DRAWN L.H.F.	INSP. [signature]	RECOMMENDED [signature]
	W. a. Waldorf	APPROVED [signature]
10-F-5	DENVER, COLO. DATE NOV. 1949	517-A-5



General Plan and Elevation
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM

Colorado A & M College
Hydraulic Laboratory
Fort Collins Dec. 1949

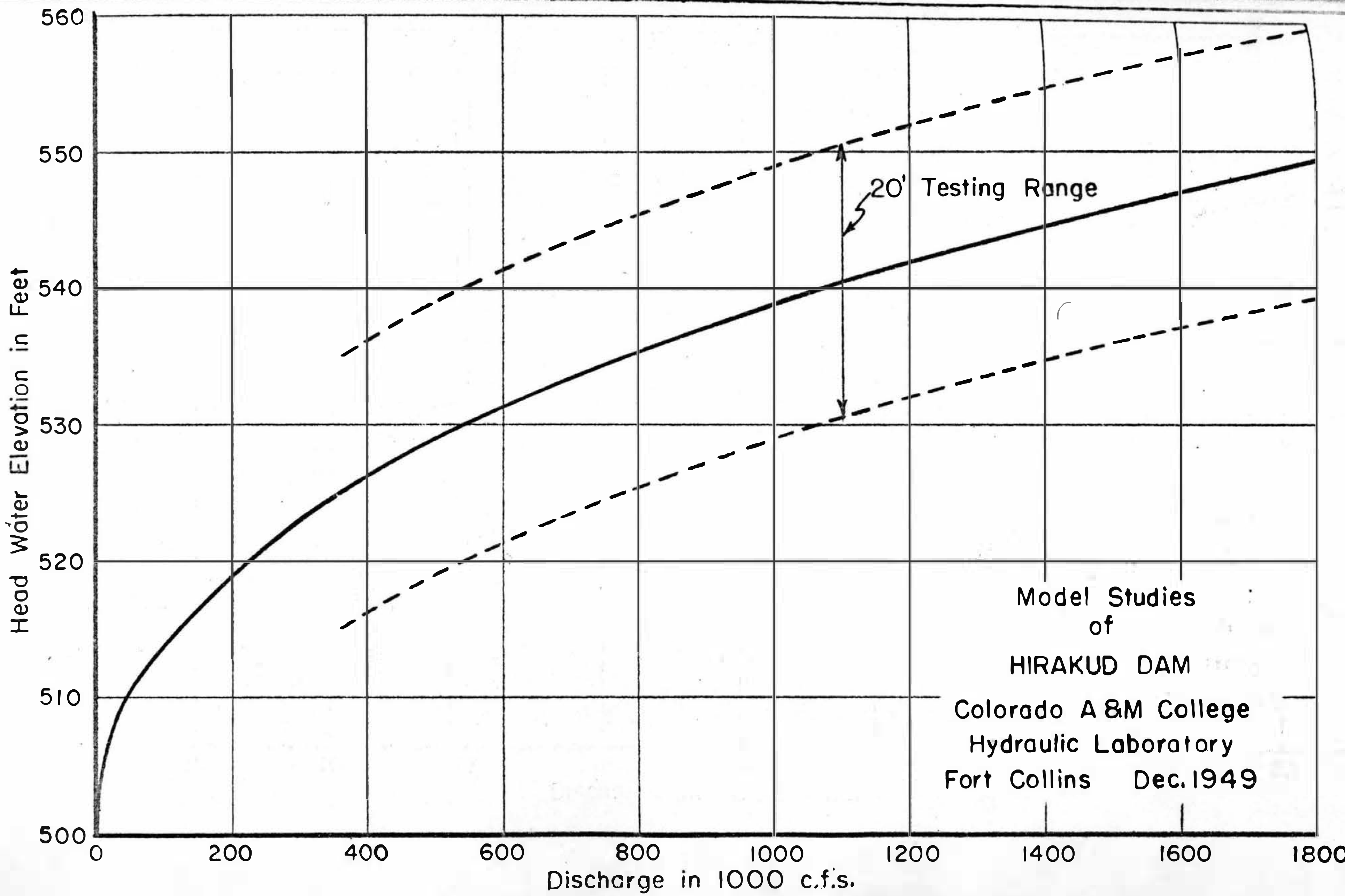


Sluice Layout, Piezometer Location, and Pressure Distribution

Sluiceway - Original Design

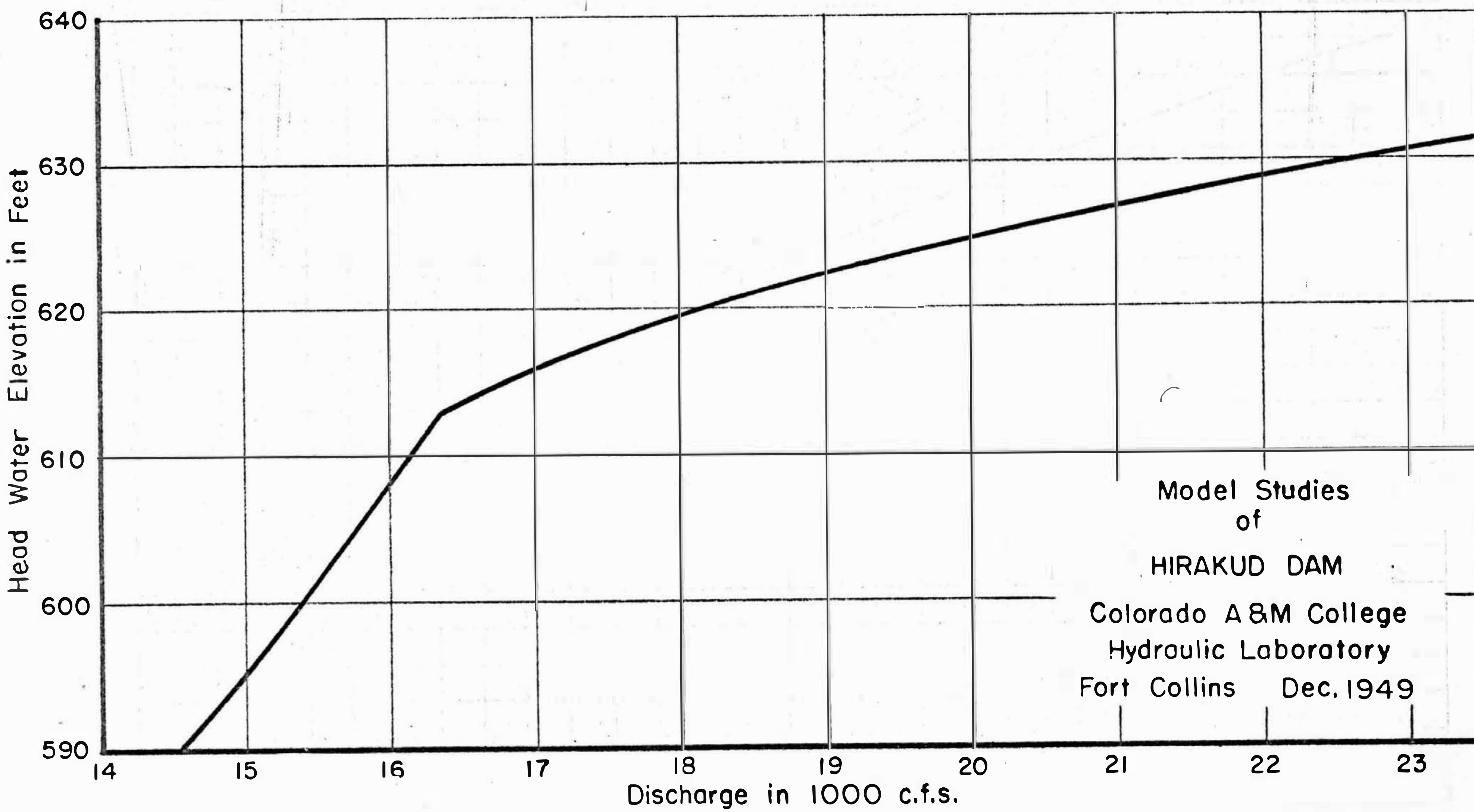
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec. 1949



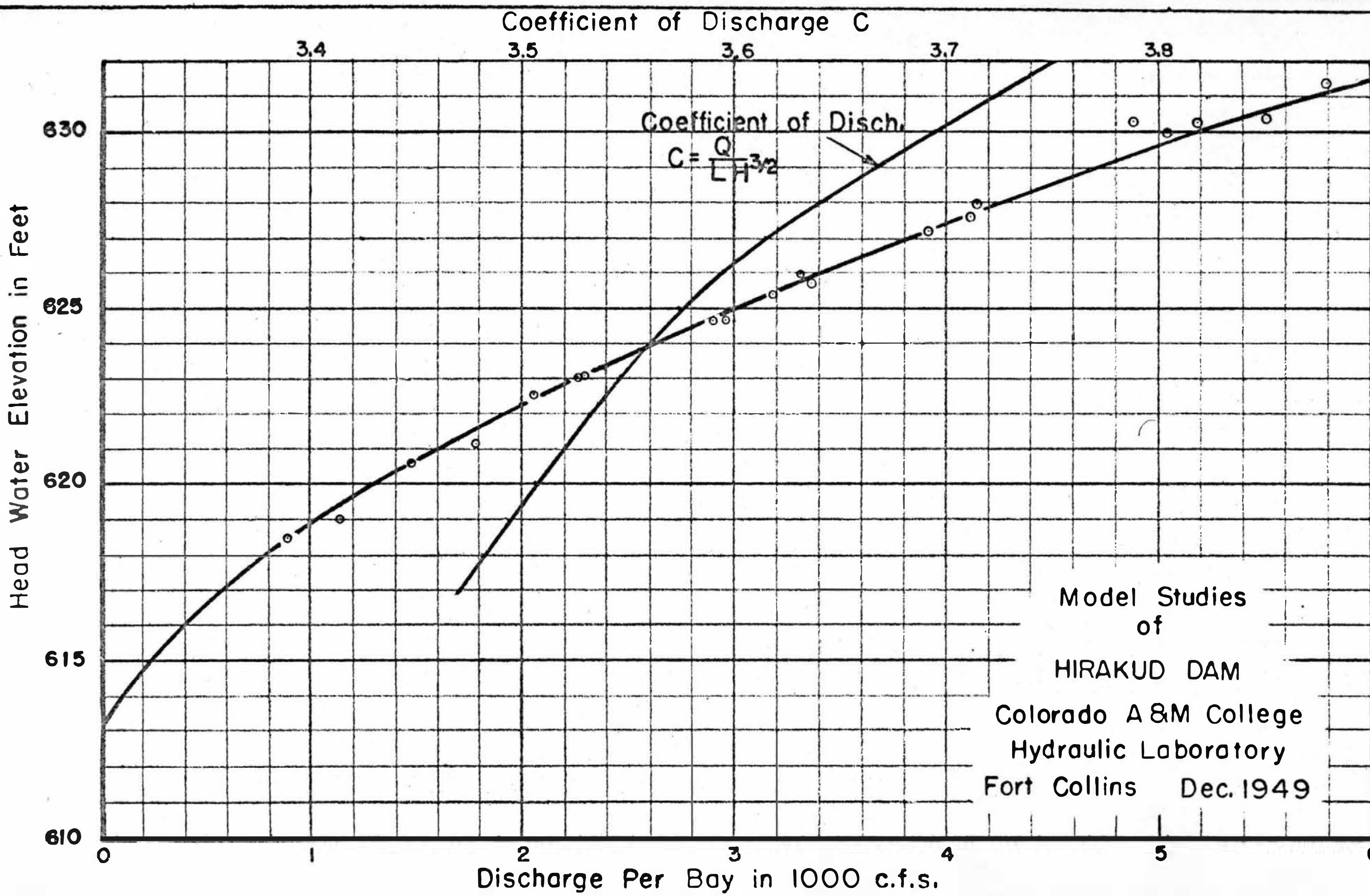
Tailwater Rating Curve

Fig. 3



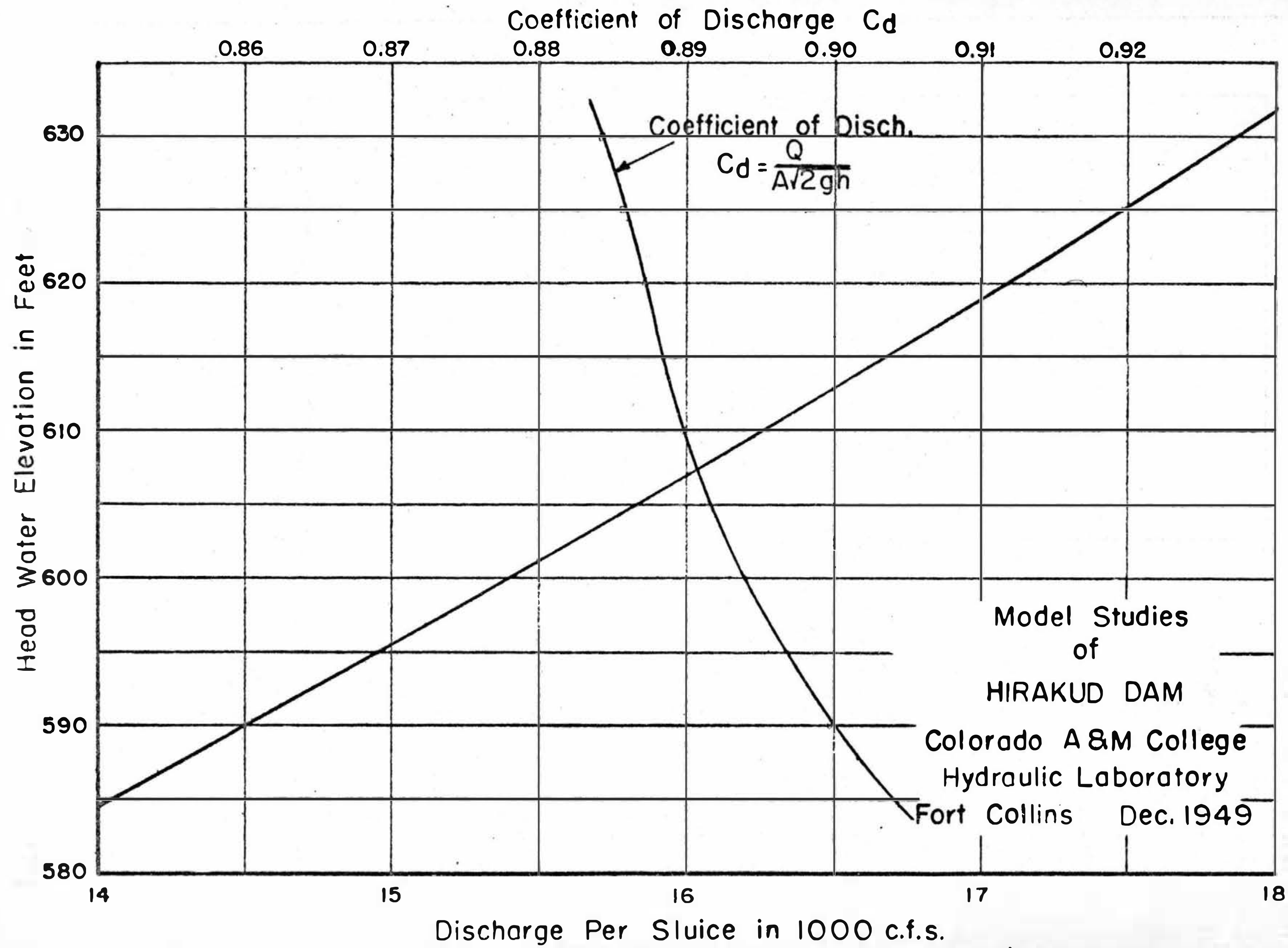
Variation of Discharge with Head
Overfall Spillway and Sluice - Original Design
1:40 Service Spillway Model

Fig. 4



Variation of Discharge and Discharge Coefficient with Head
Overfall Spillway - Original Design
1:40 Service and Auxiliary Spillway Model

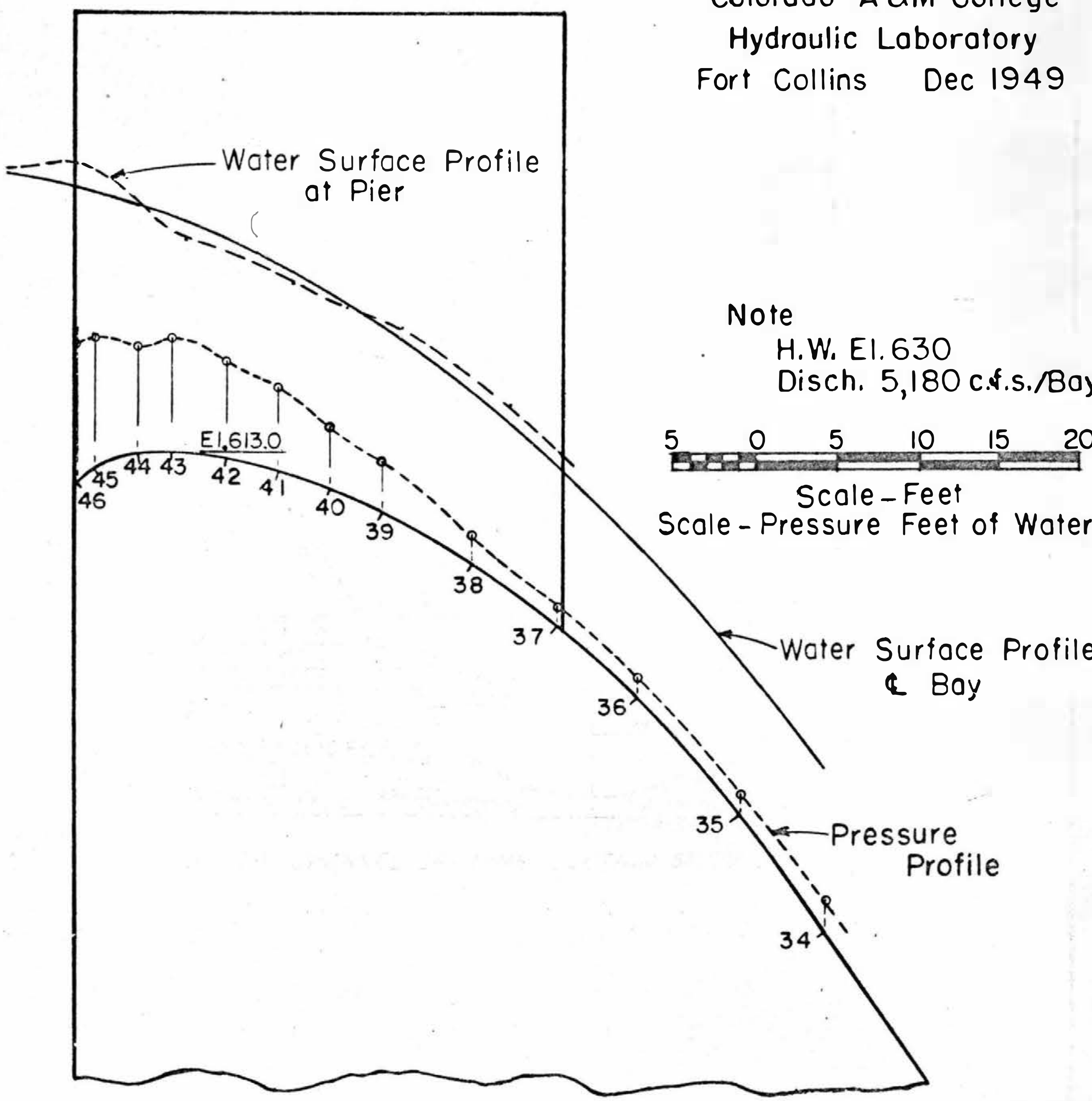
Fig. 5



Variation of Discharge and Discharge Coefficient with Head
Sluiceway - Original Design - 1:40 Service Spillway Model

Fig. 6

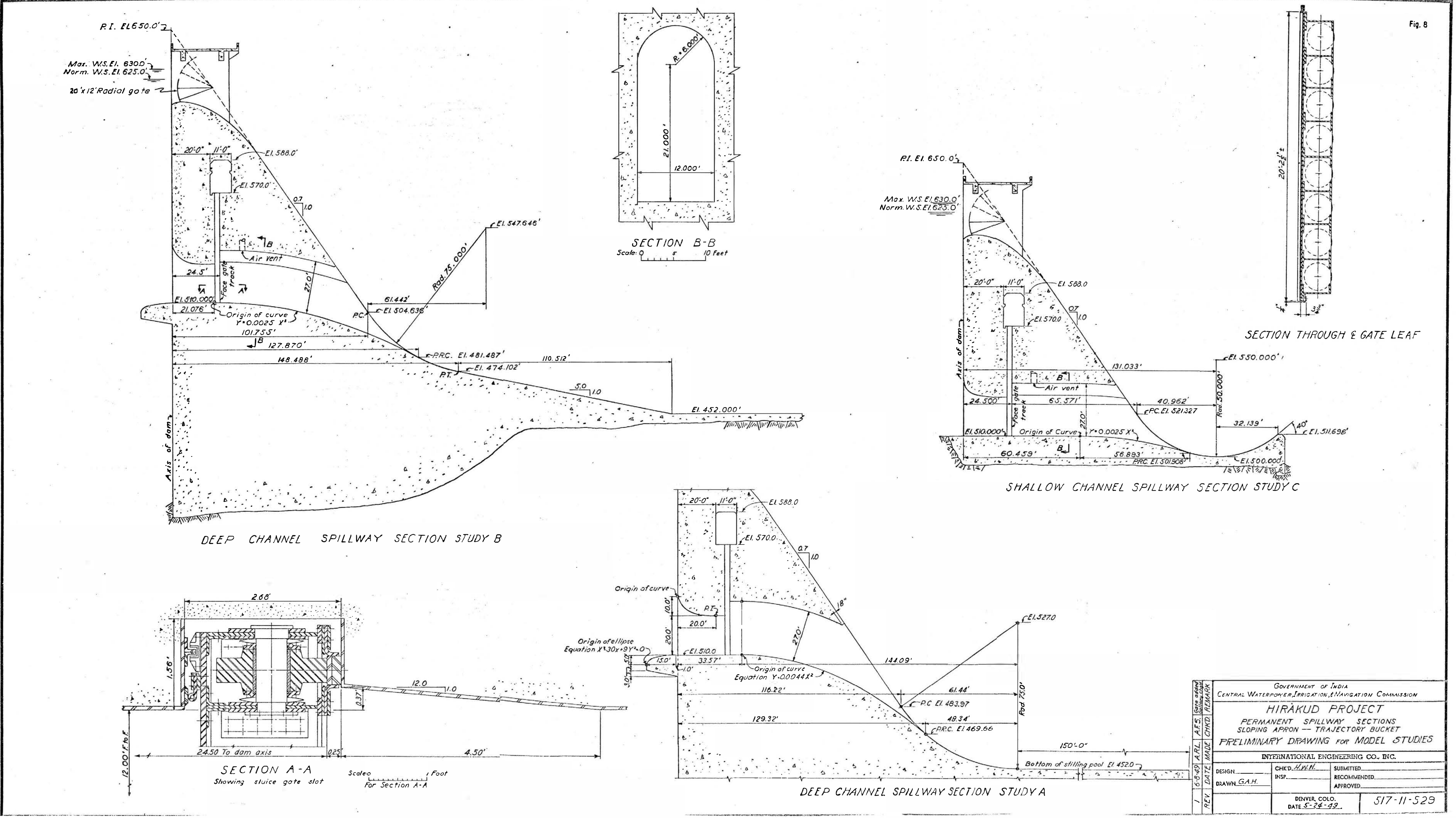
Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949



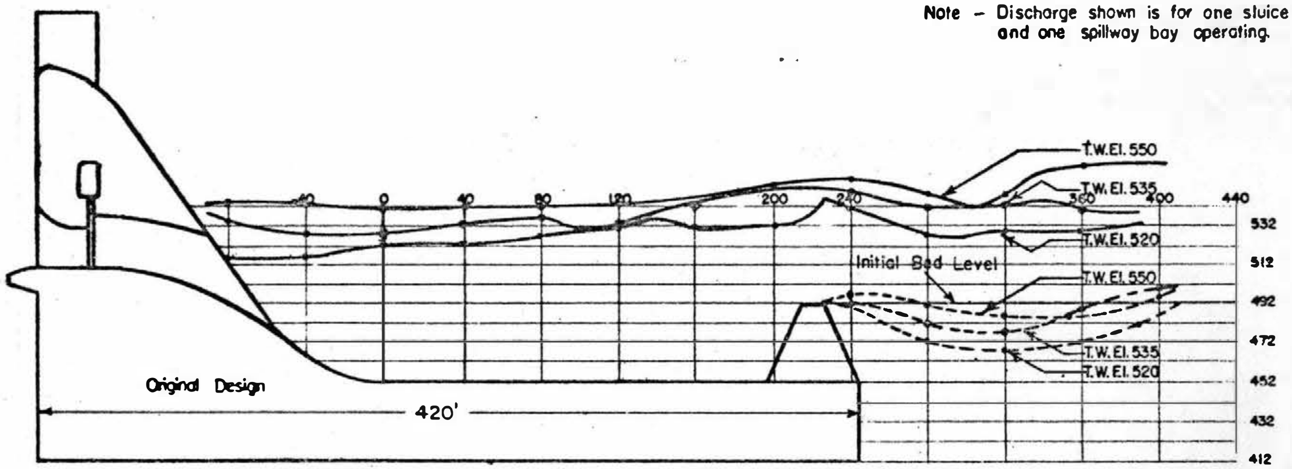
Piezometer Location, Pressure Distribution
and Water Surface Profile
Overfall Spillway Crest - Original Design
1:40 Service and Auxiliary Spillway Model

Fig. 7

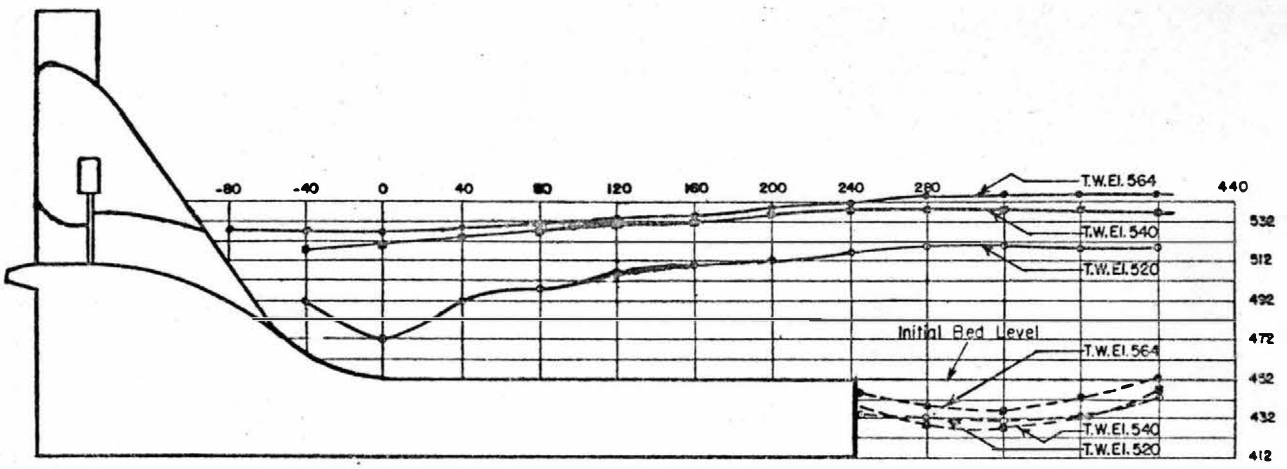
Fig. 8


P.I. El. 650.0'
Max. W.S. El. 630.0'
Norm. W.S. El. 625.0'
20'x12' Radial gate
El. 588.0'
20'-0"
11'-0"
El. 570.0'
0.7
1.0
Air vent
Face gate track
24.5'
27.0'
Rad. 75.000'
El. 547.646'
61.442'
El. 504.638'
P.C.
El. 510.000
21.076'
Origin of curve Y=0.0025 X²
101.755'
127.870'
P.R.C. El. 481.487'
148.488'
110.512'
P.T.
El. 474.102'
5.0
1.0
El. 452.000'
Axis of dam
DEEP CHANNEL SPILLWAY SECTION STUDY B
R=6.000'
21.000'
12.000'
SECTION B-B
Scale: 0 5 10 feet
P.I. El. 650.0'
Max. W.S. El. 630.0'
Norm. W.S. El. 625.0'
El. 588.0
El. 570.0
131.033'
El. 550.000'
Rad. 50.000'
65.571'
24.500'
40.962'
P.C. El. 521.327
32.139'
40°
El. 511.698'
El. 510.000'
Origin of Curve Y=0.0025 X²
60.459'
56.893'
P.R.C. El. 501.908'
El. 500.000'
SHALLOW CHANNEL SPILLWAY SECTION STUDY C
20'-2½"±
3⅜"
SECTION THROUGH ℄ GATE LEAF
2.68'
1.56'
12.0
1.0
0.37'
24.50 To dam axis
0.25'
4.50'
12.00' F. to F.
SECTION A-A
Showing sluice gate slot
Scale 0 1 Foot
For Section A-A
El. 588.0
El. 570.0
Origin of curve
10.0'
P.T.
20.0'
20.0'
18"
27.0'
El. 527.0
Origin of ellipse
Equation X²+30x+9Y²=0
El. 510.0
15.0'
33.57'
144.09'
Origin of curve
Equation Y=0.0044X²
116.22'
61.44'
P.C. El. 483.97
Rad. 75.0'
129.32'
48.34'
P.R.C. El. 469.66
150'-0"
Bottom of stilling pool El. 452.0
DEEP CHANNEL SPILLWAY SECTION STUDY A
1 | 6-8-49 | A.R.L. | A.F.S. | Gate added
REV. | DATE | MADE | CHK'D | REMARK
GOVERNMENT OF INDIA
CENTRAL WATERPOWER, IRRIGATION, & NAVIGATION COMMISSION
HIRAKUD PROJECT
PERMANENT SPILLWAY SECTIONS
SLOPING APRON — TRAJECTORY BUCKET
PRELIMINARY DRAWING FOR MODEL STUDIES
INTERNATIONAL ENGINEERING CO. INC.
DESIGN
DRAWN G.A.H.
CHK'D. H.W.H.
INSP.
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLO.
DATE 5-24-49
517-11-529

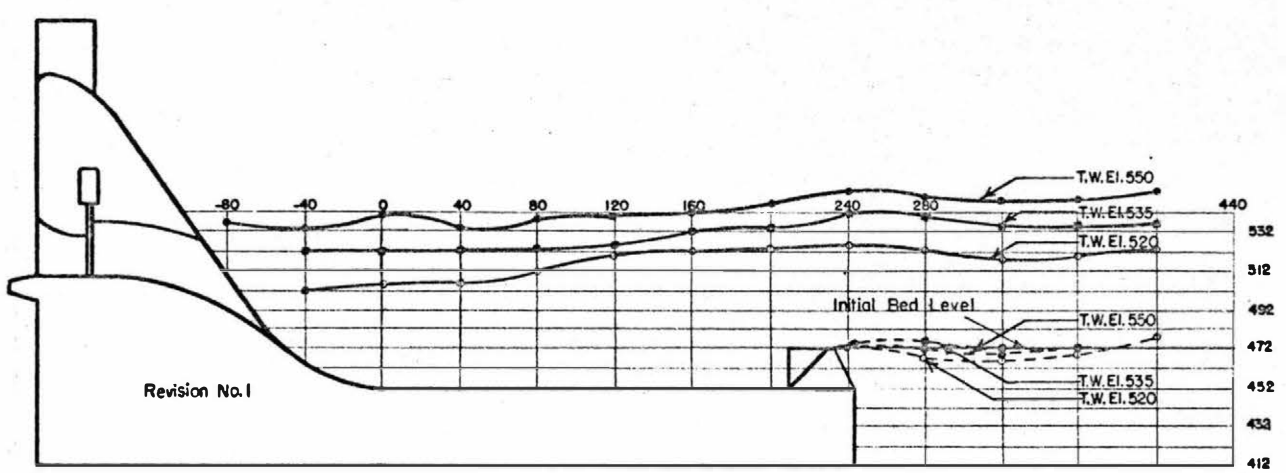
Note – Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 625.0 Q = 20,120 c.f.s.



H.W. El. 630.2 Q = 22,650 c.f.s.



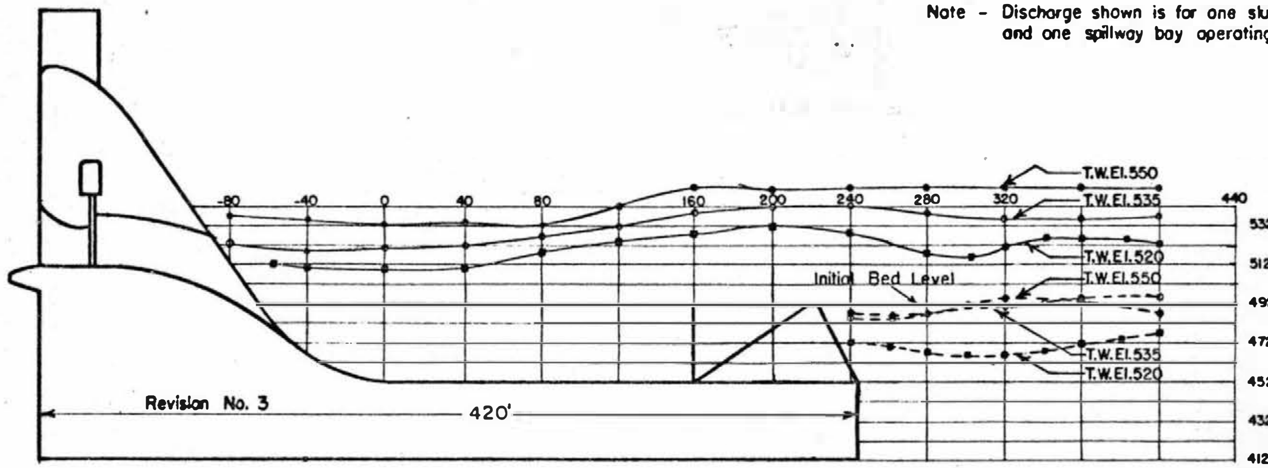
H.W. El. 628.5 Q = 21,800 c.f.s.

Water Surface and Erosion Profiles
Effect of Sill Design
Apron El. 452 - Overall Length 420 ft.
1:40 Service Spillway Model

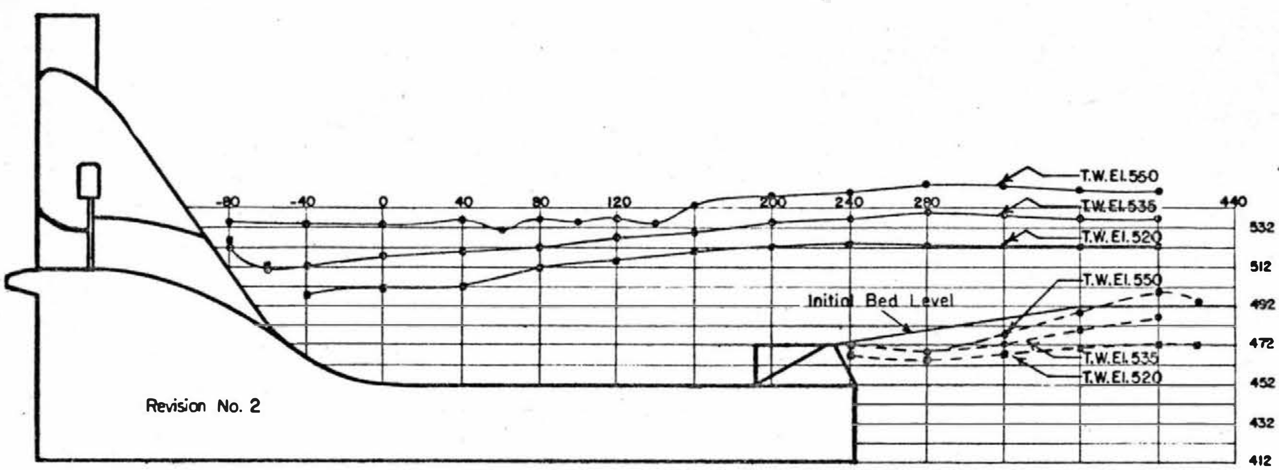
Model Studies
of
HIRAKUD DAM
Colorado A & M College
Hydraulic Laboratory
Fort Collins Dec 1949

Fig. 10

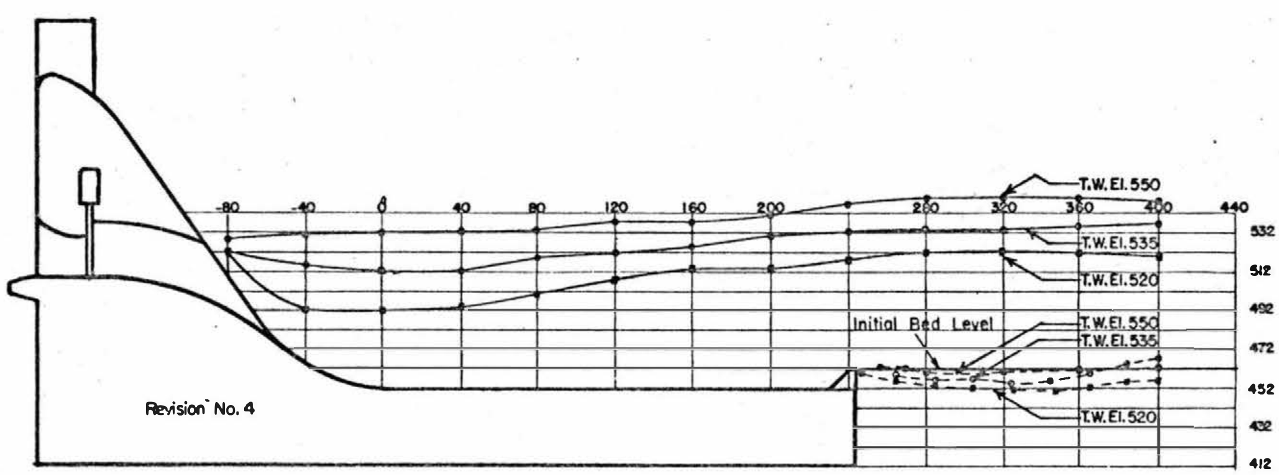
Note - Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 630.2 Q = 22,770 c.f.s.



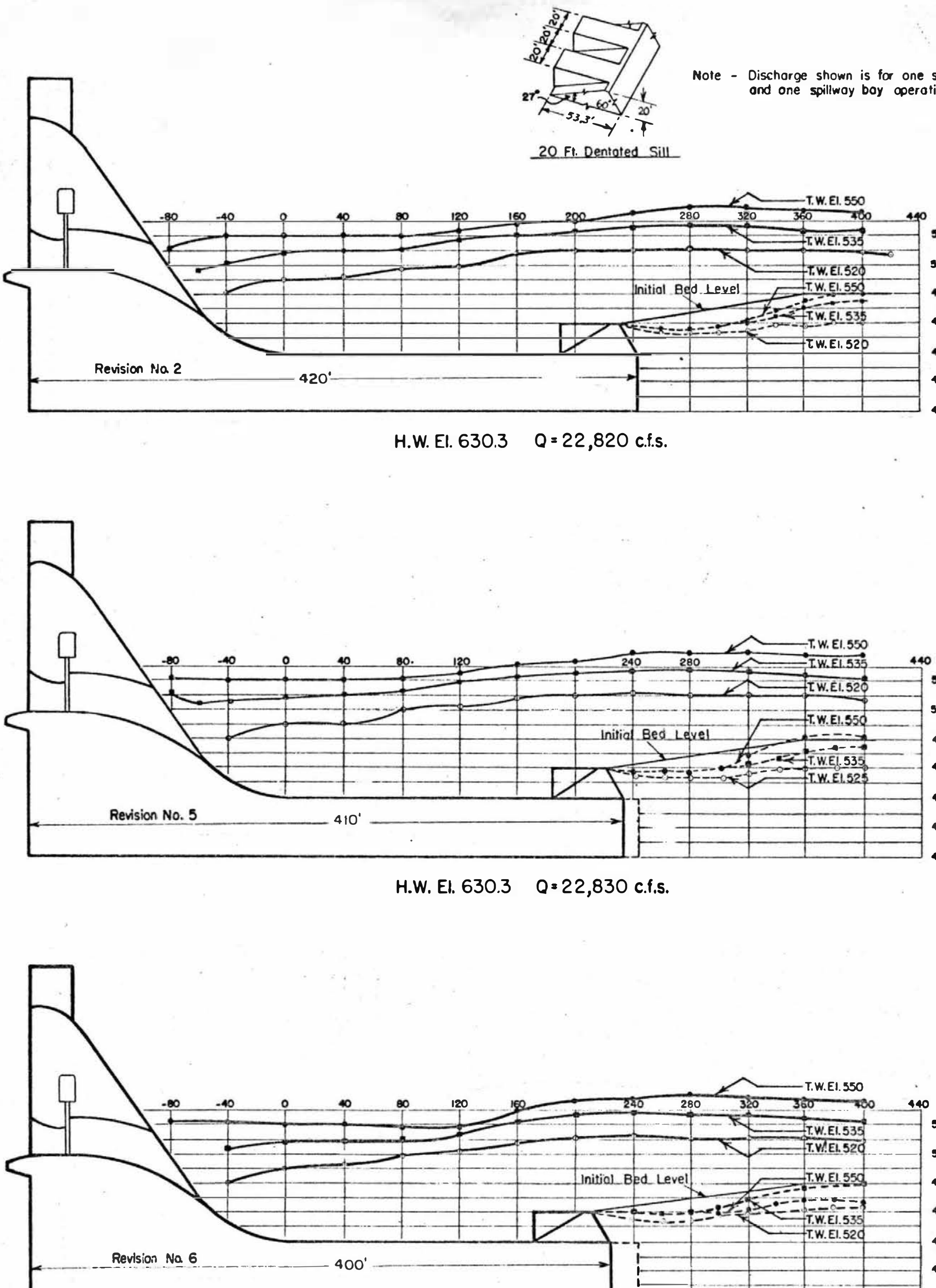
H.W. El. 629.8 Q = 22,550 c.f.s.



H.W. El. 630.2 Q = 22,770 c.f.s.

Water Surface and Erosion Profiles
Effect of Sill Design
Apron El. 452 - Overall Length 420 ft.
1:40 Service Spillway Model

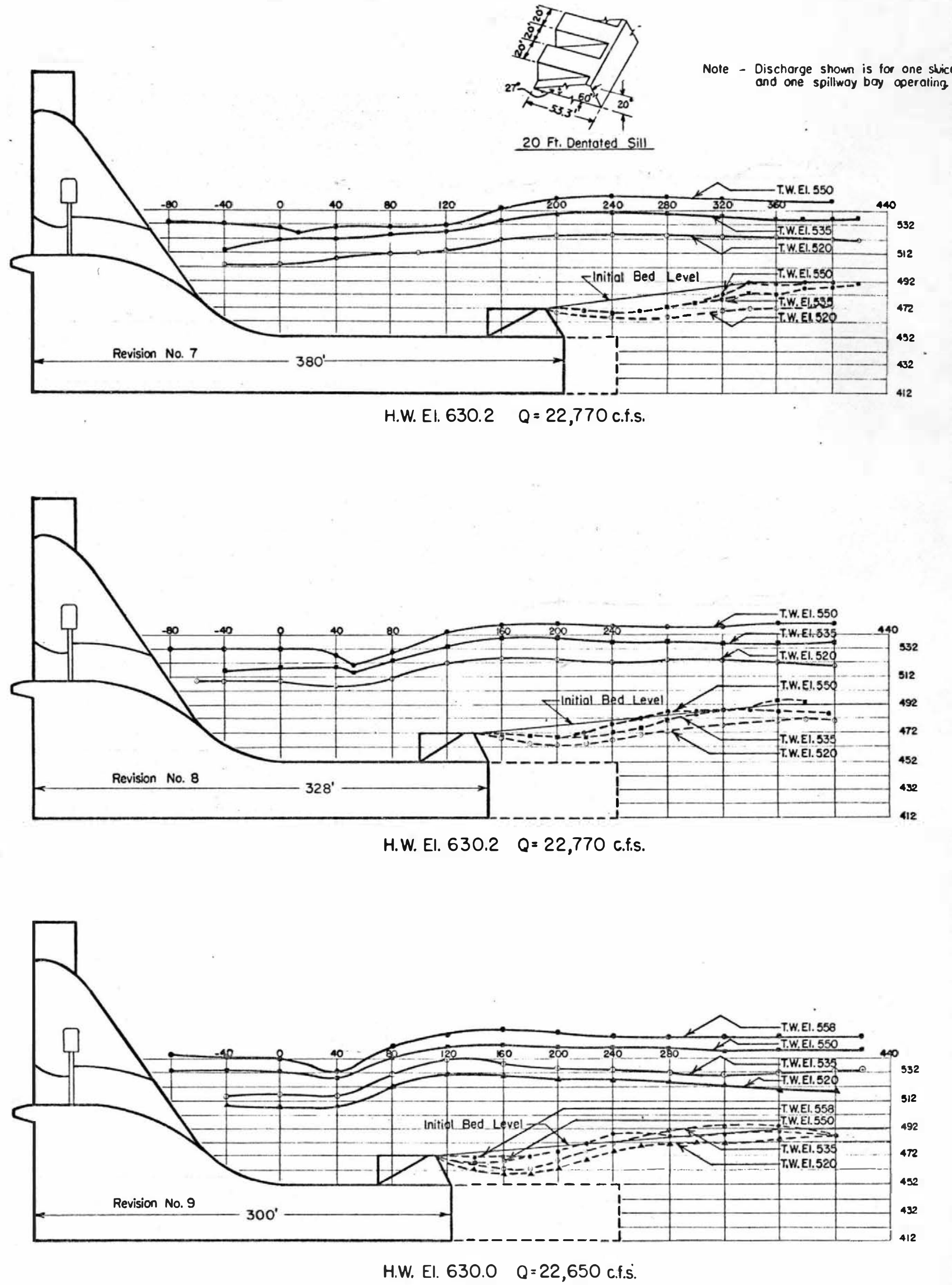
Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec. 1949



Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 452 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

Fig. 12

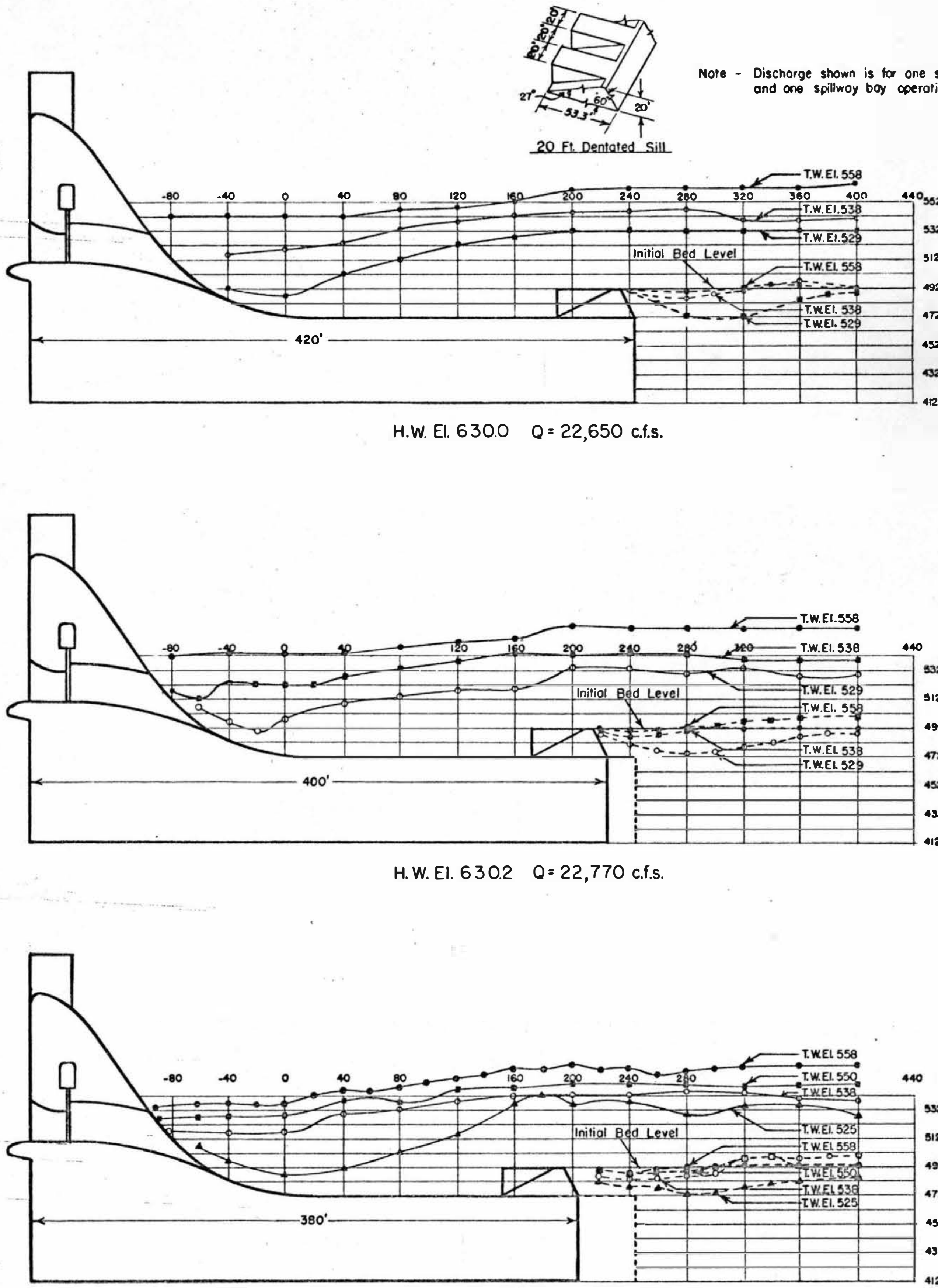


Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 452 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

Fig. 13

Note - Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 630.0 Q = 22,650 c.f.s.

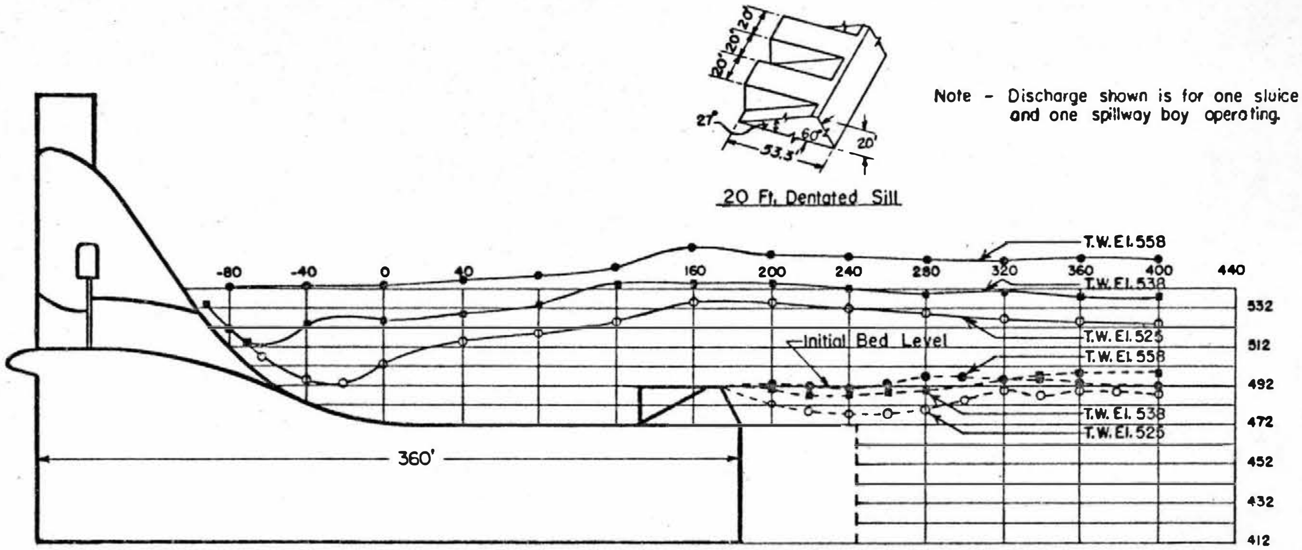
H.W. El. 630.2 Q = 22,770 c.f.s.

H.W. El. 630.4 Q = 22,890 c.f.s.

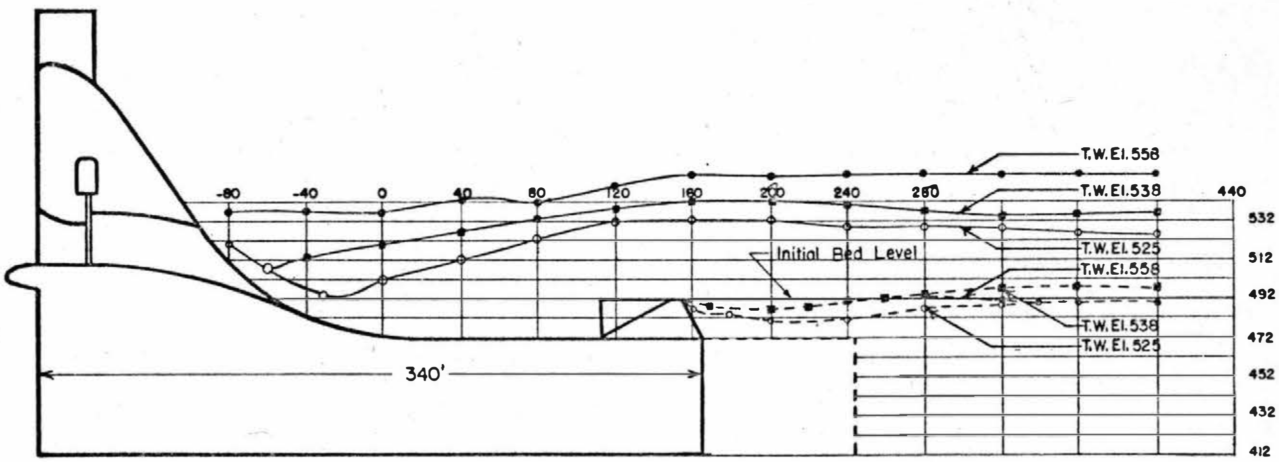
Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 472 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

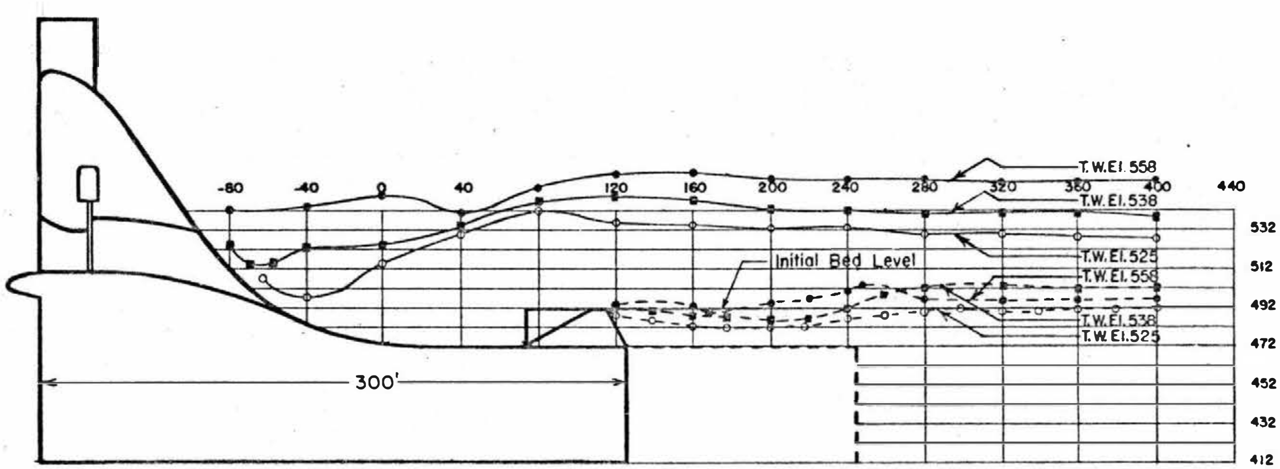
Fig. 14



H.W. El. 630.2 Q = 22,770 c.f.s.



H. W. El. 630.2 Q = 22,770 c.f.s.

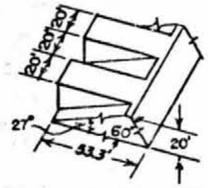


H.W. El. 630.3 Q = 22,830 c.f.s.

Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 472 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

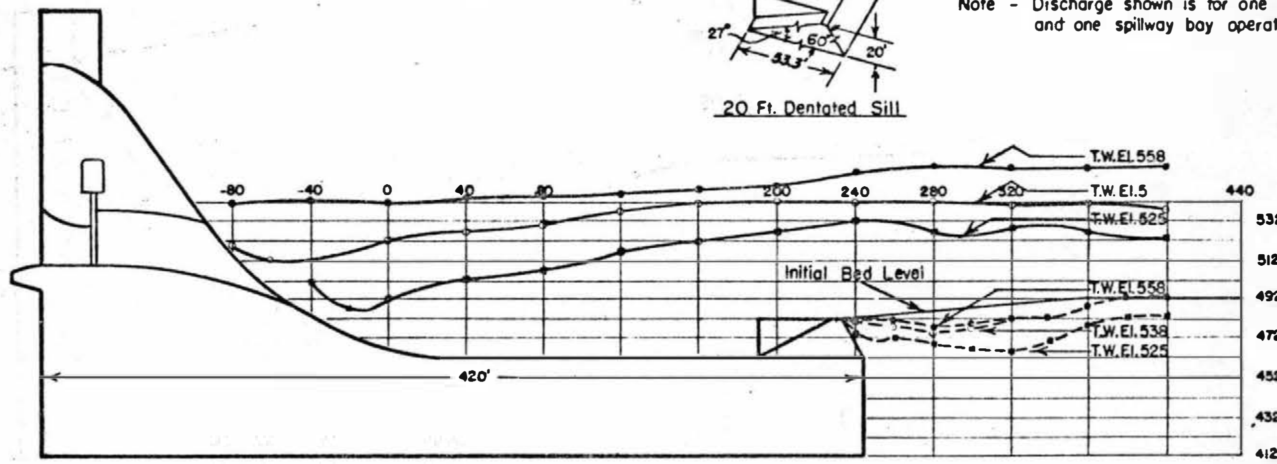
Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

Fig. 15

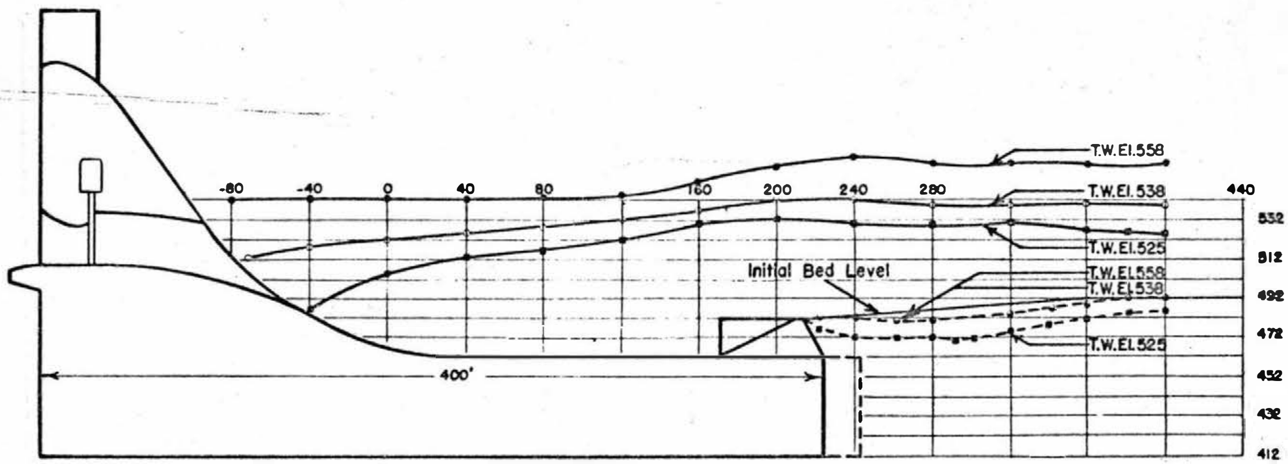


20 Ft. Dentated Sill

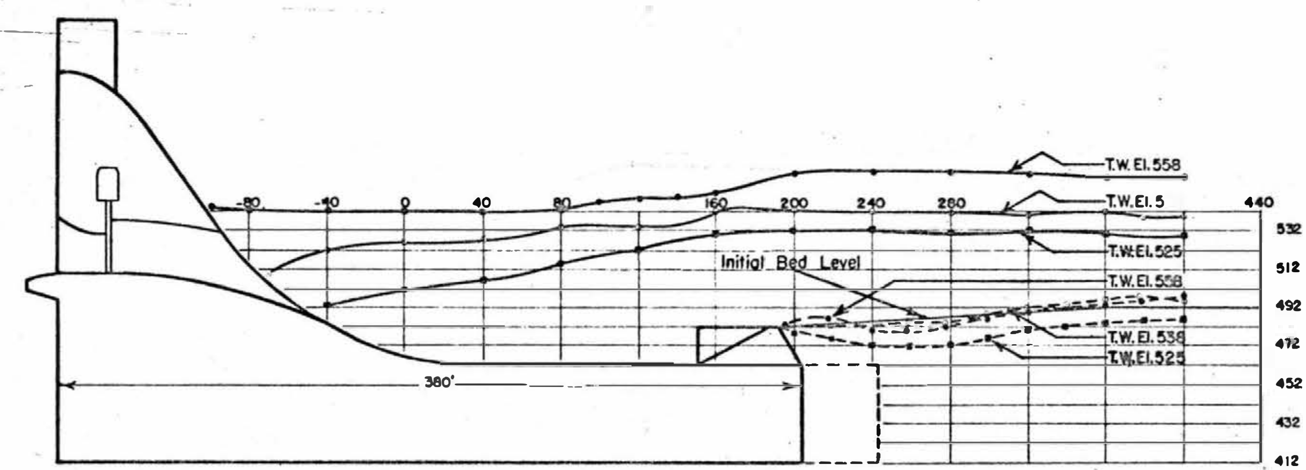
Note - Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 630.6 Q = 23,000 c.f.s.



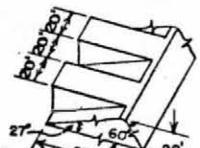
H.W. El. 629.8 Q = 22,550 c.f.s.



H.W. El. 630.5 Q = 22,950 c.f.s.

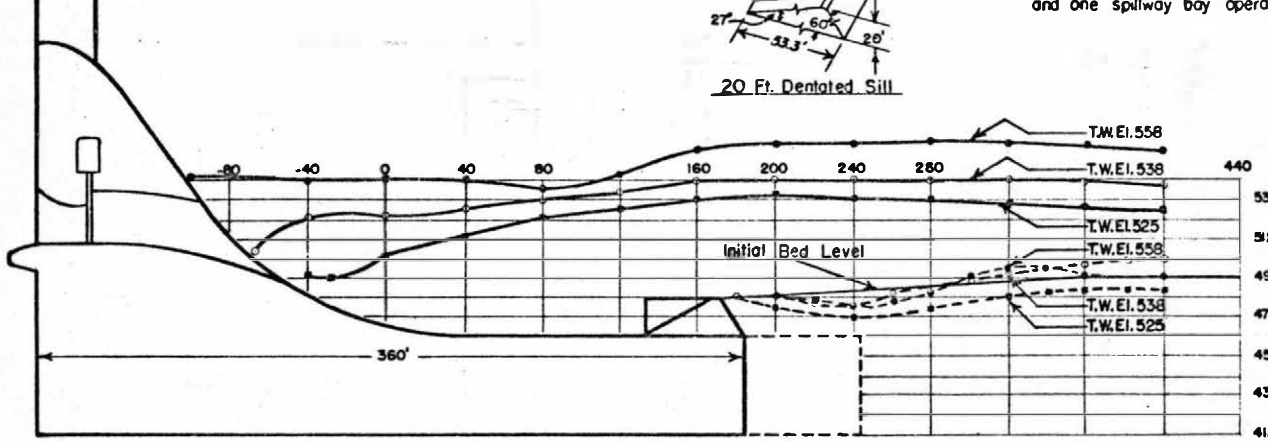
Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 462 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

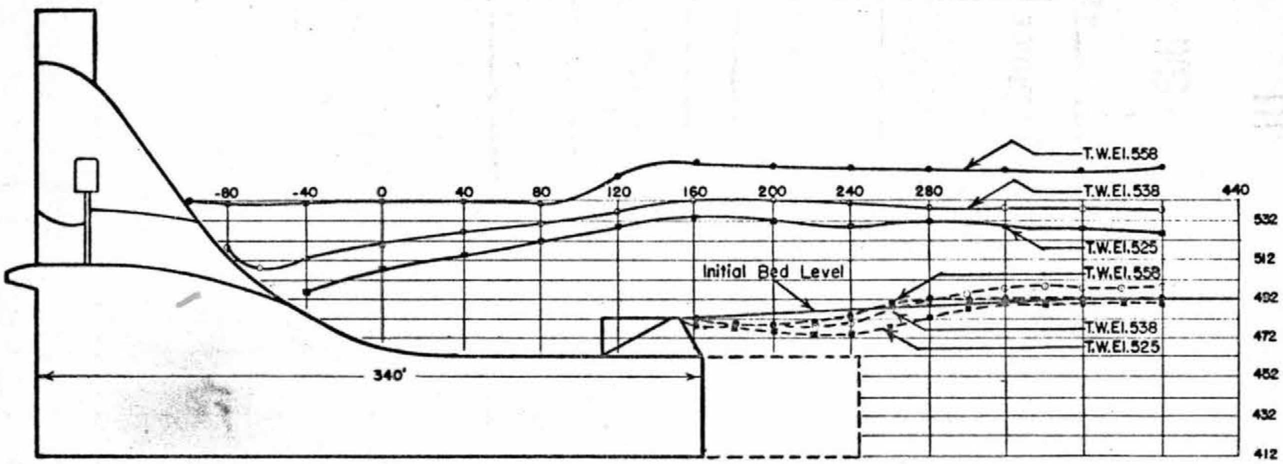


20 Ft. Dentated Sill

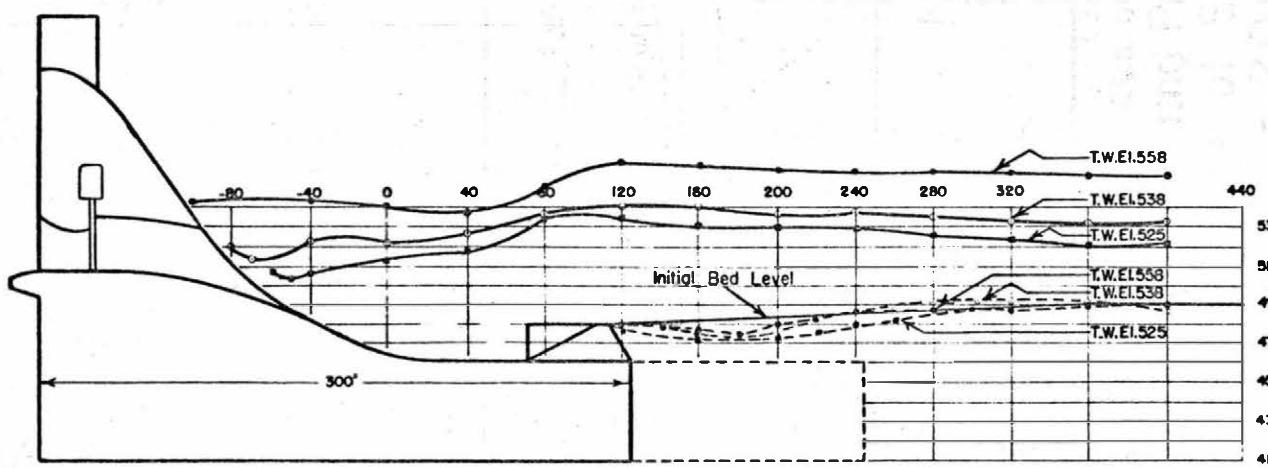
Note - Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 630.7 Q = 23,060 c.f.s.



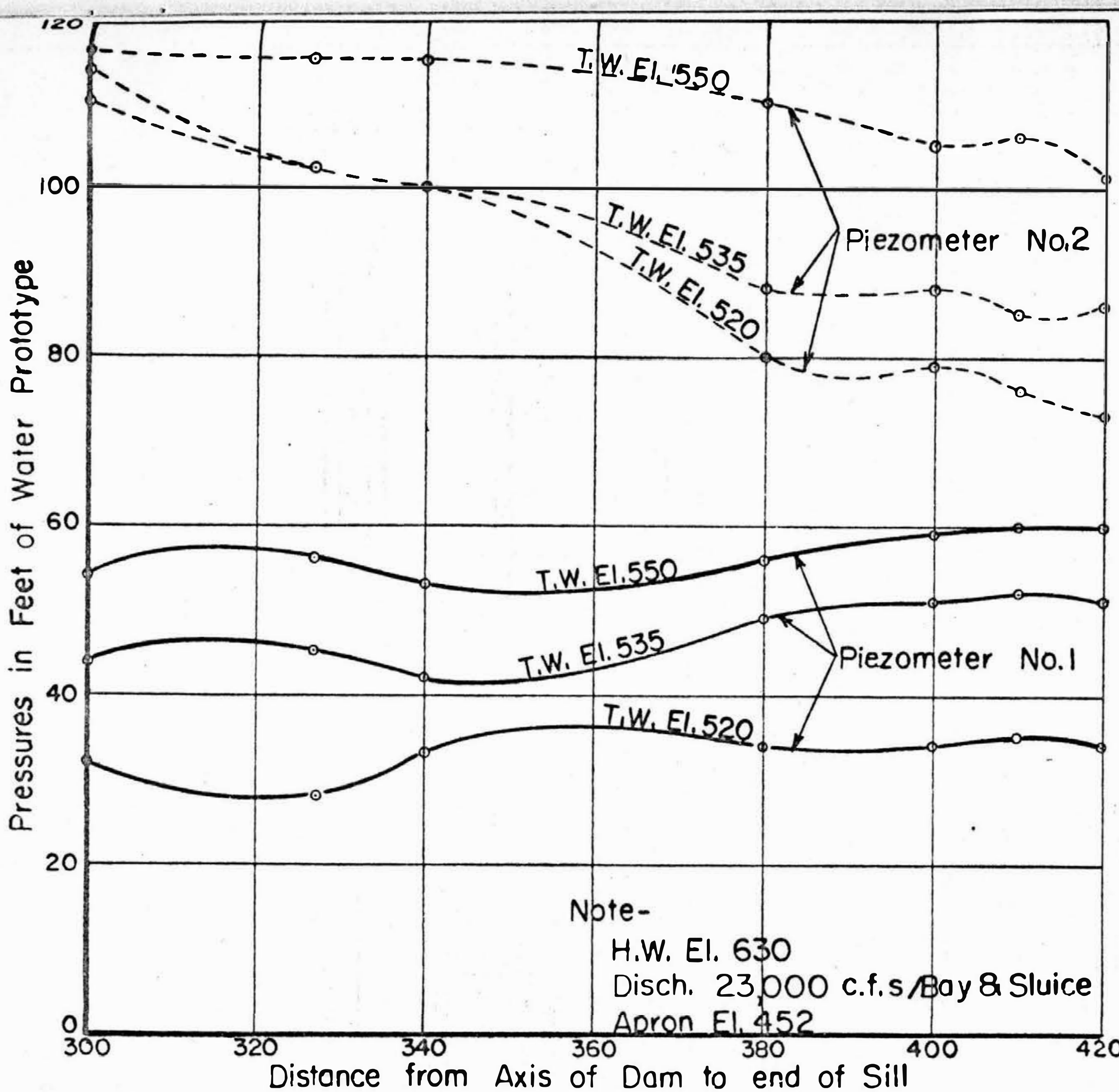
H.W. El. 630.2 Q = 22,770 c.f.s.



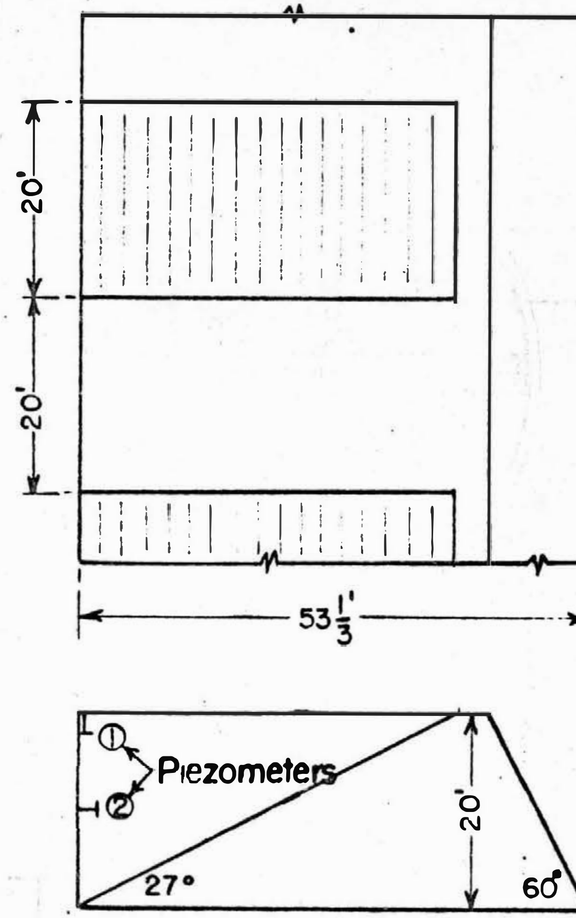
H.W. El. 630.2 Q = 22,770 c.f.s.

Water Surface and Erosion Profiles
Effect of Apron Length
Apron El. 462 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949



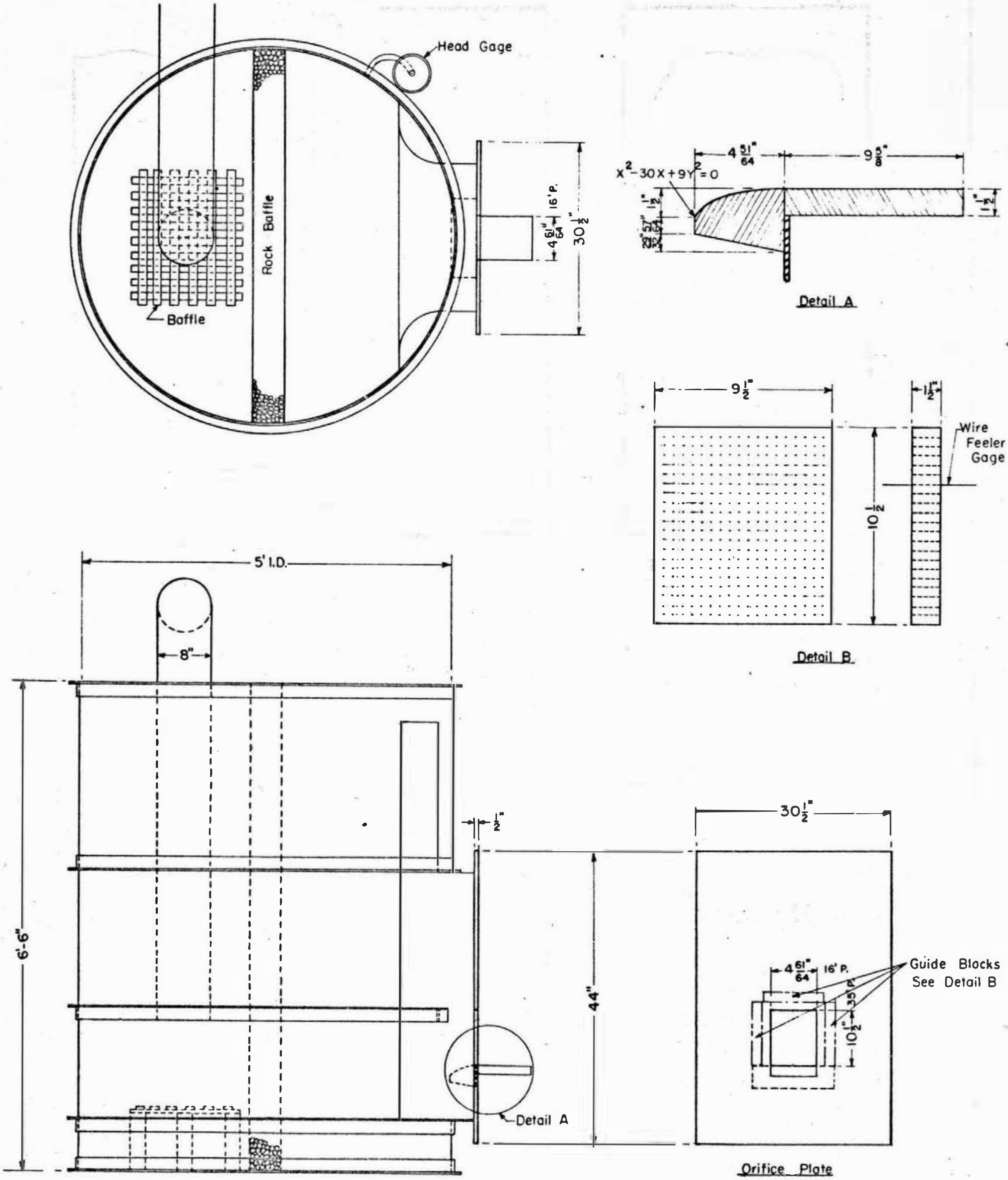
Piezometer Location and Pressure Distribution on Sill
Effect of Sill Location
Apron El. 452 - 20 ft. 27° Dentated Sill
1:40 Service Spillway Model



Model Studies
of
HIRAKUD DAM

Colorado A & M College
Hydraulic Laboratory
Fort Collins Dec 1949

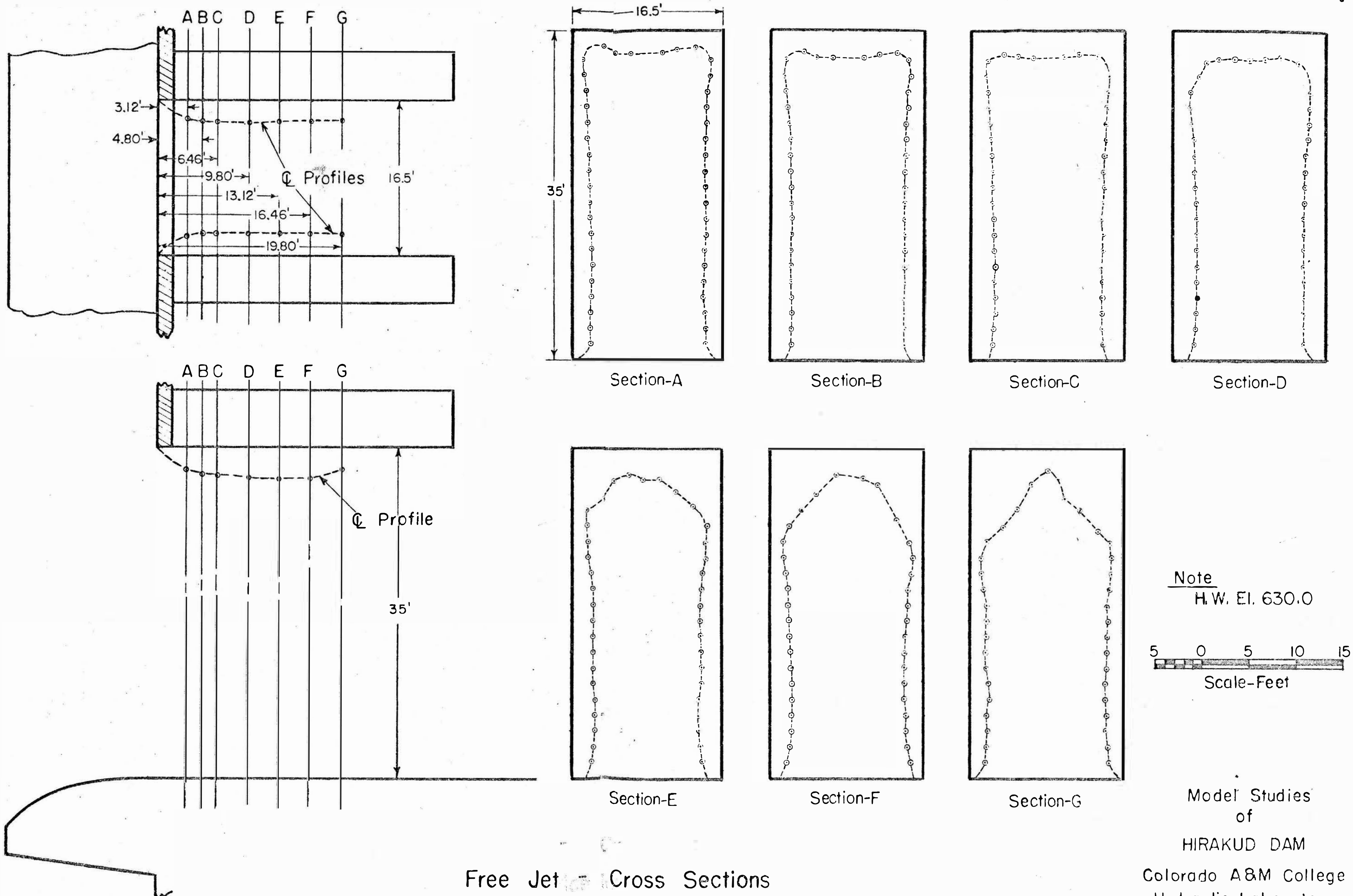
Fig. 17



General Plan and Elevation
1:40 Sluice Inlet Model

Model Studies
of
HIRAKUD DAM
Colorado A & M College
Hydraulic Laboratory
Fort Collins Dec. 1949

Fig. 19

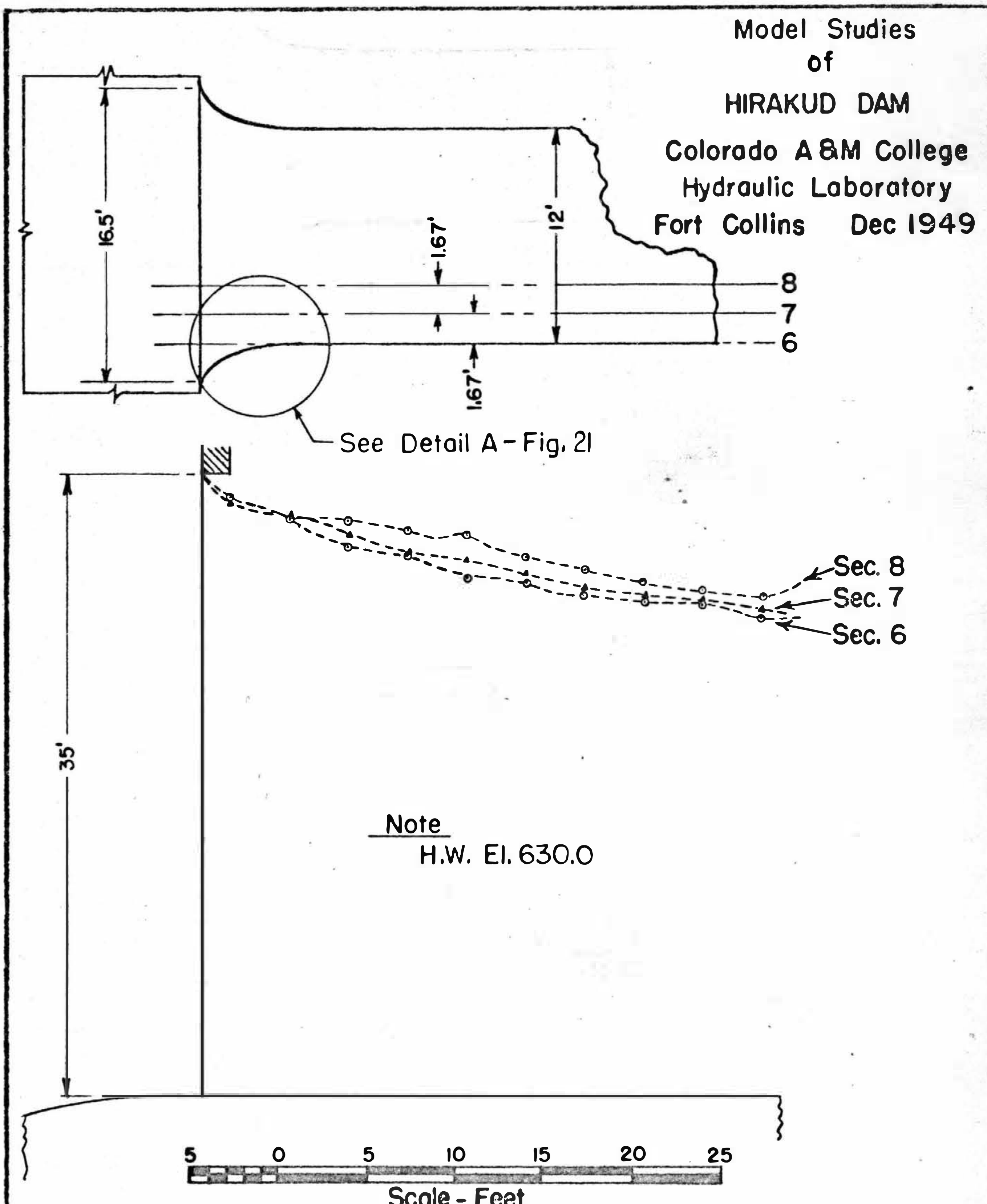


Free Jet - Cross Sections

1:40 Sluice Inlet Model

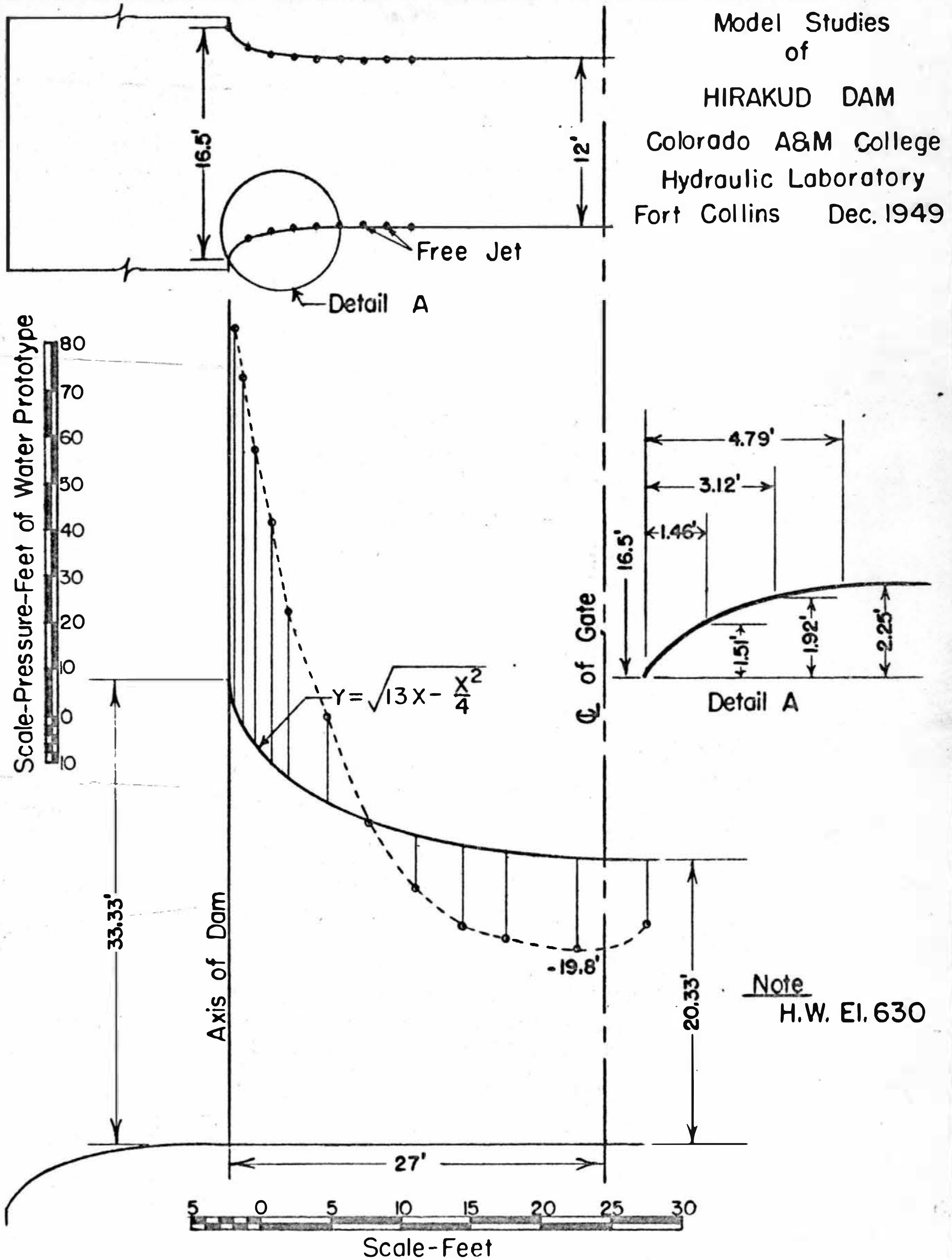
Model Studies
of
HIRAKUD DAM

Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec. 1949



Top Profile
Sidewall Revision No.2 - Inlet 35 ft. High
1:40 Sluice Inlet Model

Fig. 20



Piezometer Location and Pressure Distribution
Sidewall Revision No.2 - Roof Revision No.1
1:40 Sluice Inlet Model

Fig. 21

Model Studies
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HIRAKUD DAM
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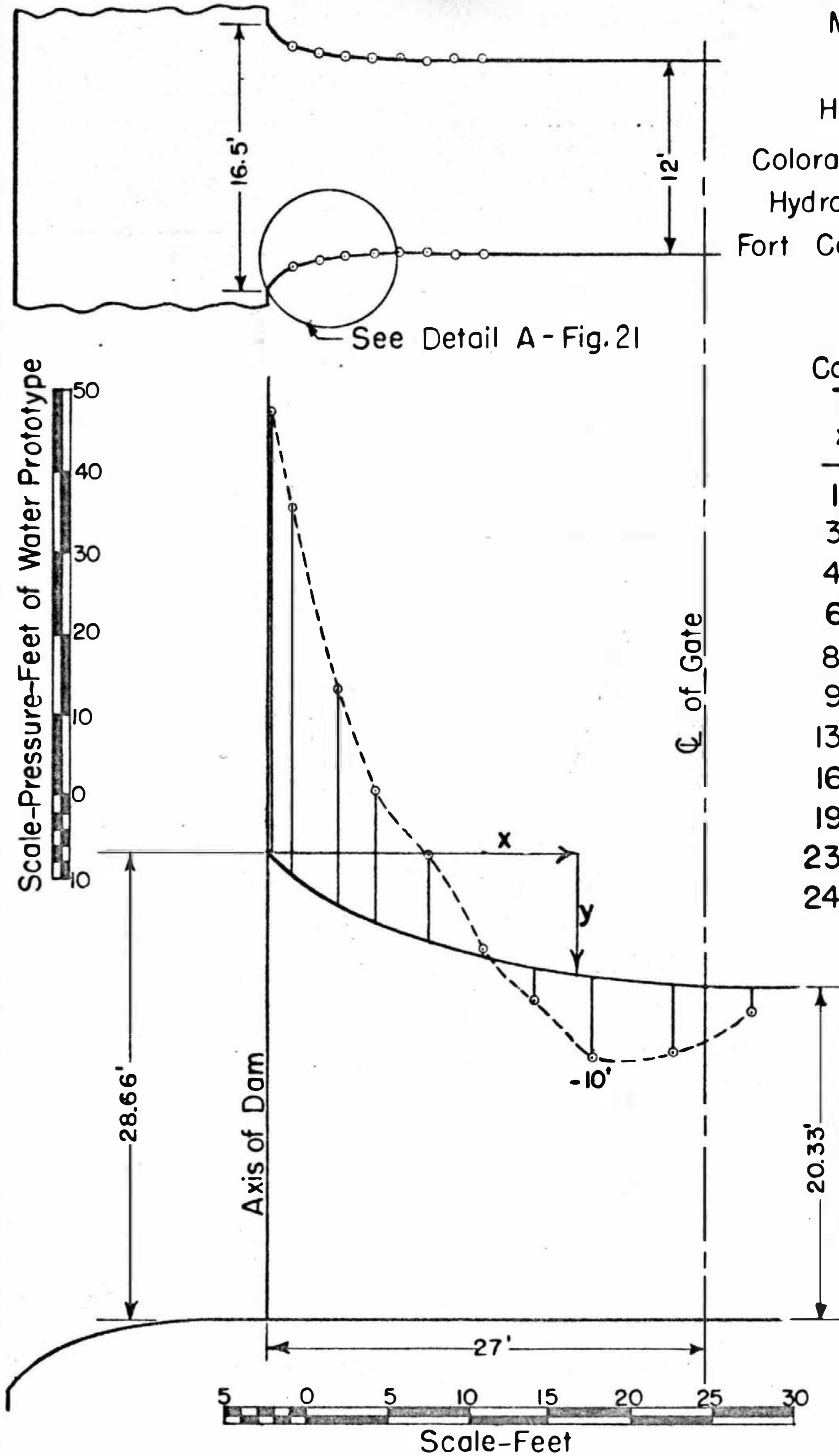

Coordinates of Top Curve

x	y
1.46'	1.60'
3.12'	2.67'
4.79'	3.60'
6.46'	4.33'
8.12'	5.00'
9.79'	5.53'
13.12'	6.53'
16.46'	7.27'
19.79'	7.83'
23.12'	8.20'
24.79'	8.33'

Note
H.W. El. 626.4

Piezometer Location and Pressure Distribution
Sidewall Revision No.2 - Roof Revision No.2
1:40 Sluice Inlet Model

Fig. 22

Model Studies
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HIRAKUD DAM

Colorado A&M College
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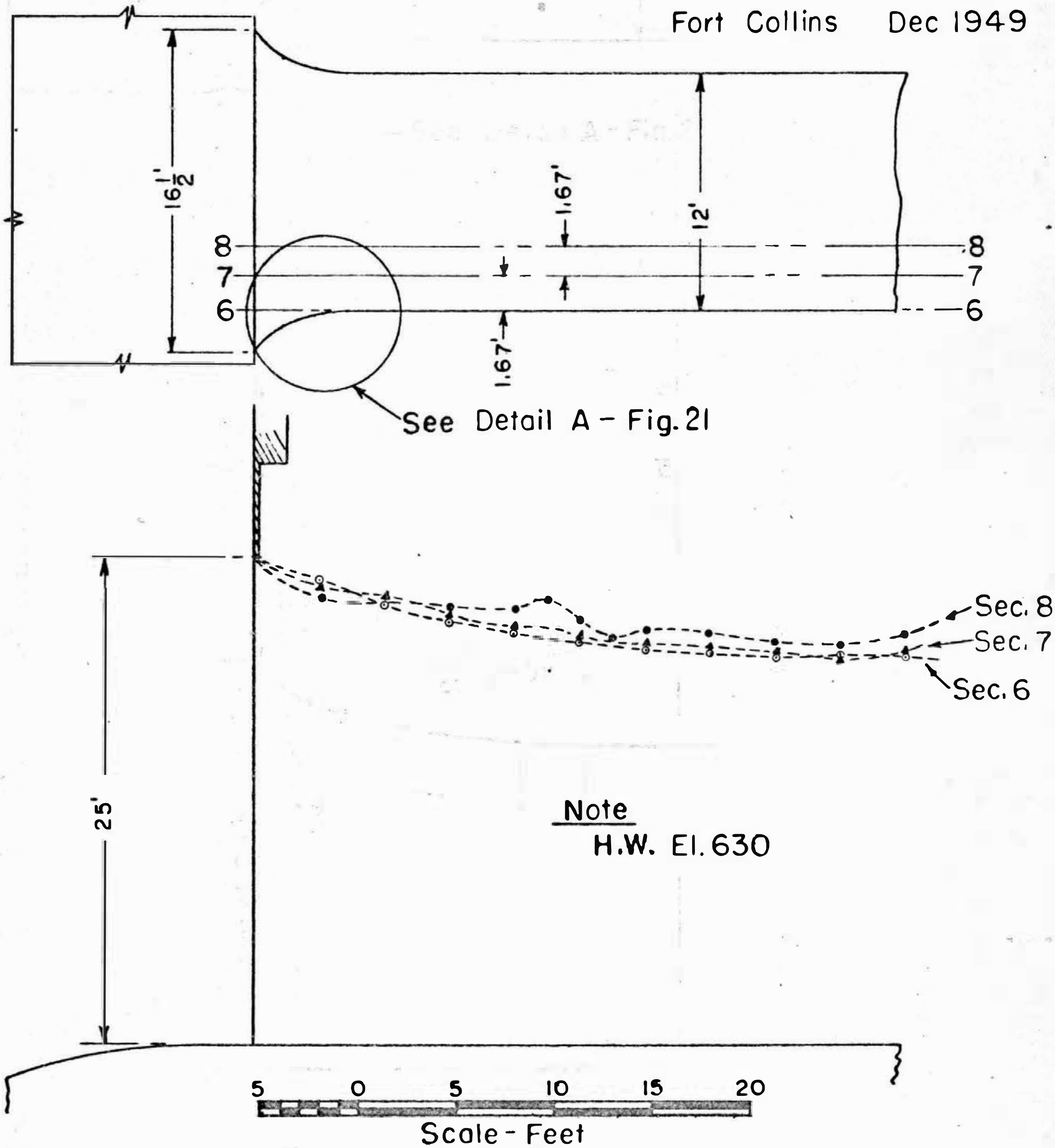
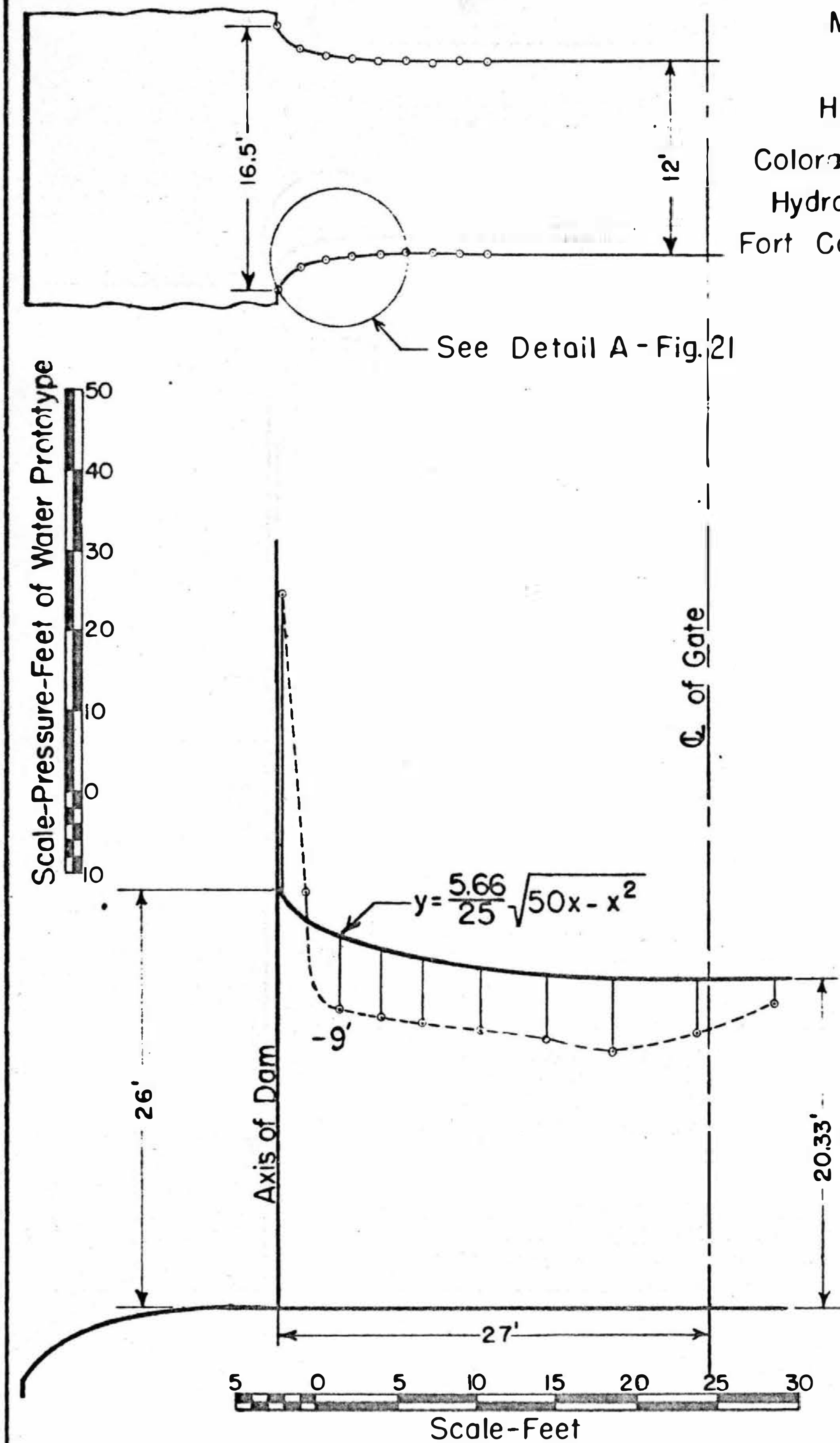


Top Profile
Sidewall Revision No. 2 - Inlet 25 ft. High
1:40 Sluice Inlet Model

Fig. 23

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949



Note
H.W. El. 630.3

Piezometer Location and Pressure Distribution
Sidewall Revision No.2 - Roof Revision No.3
1:40 Sluice Inlet Model

Fig. 24

Model Studies
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HIRAKUD DAM
Colorado A&M College
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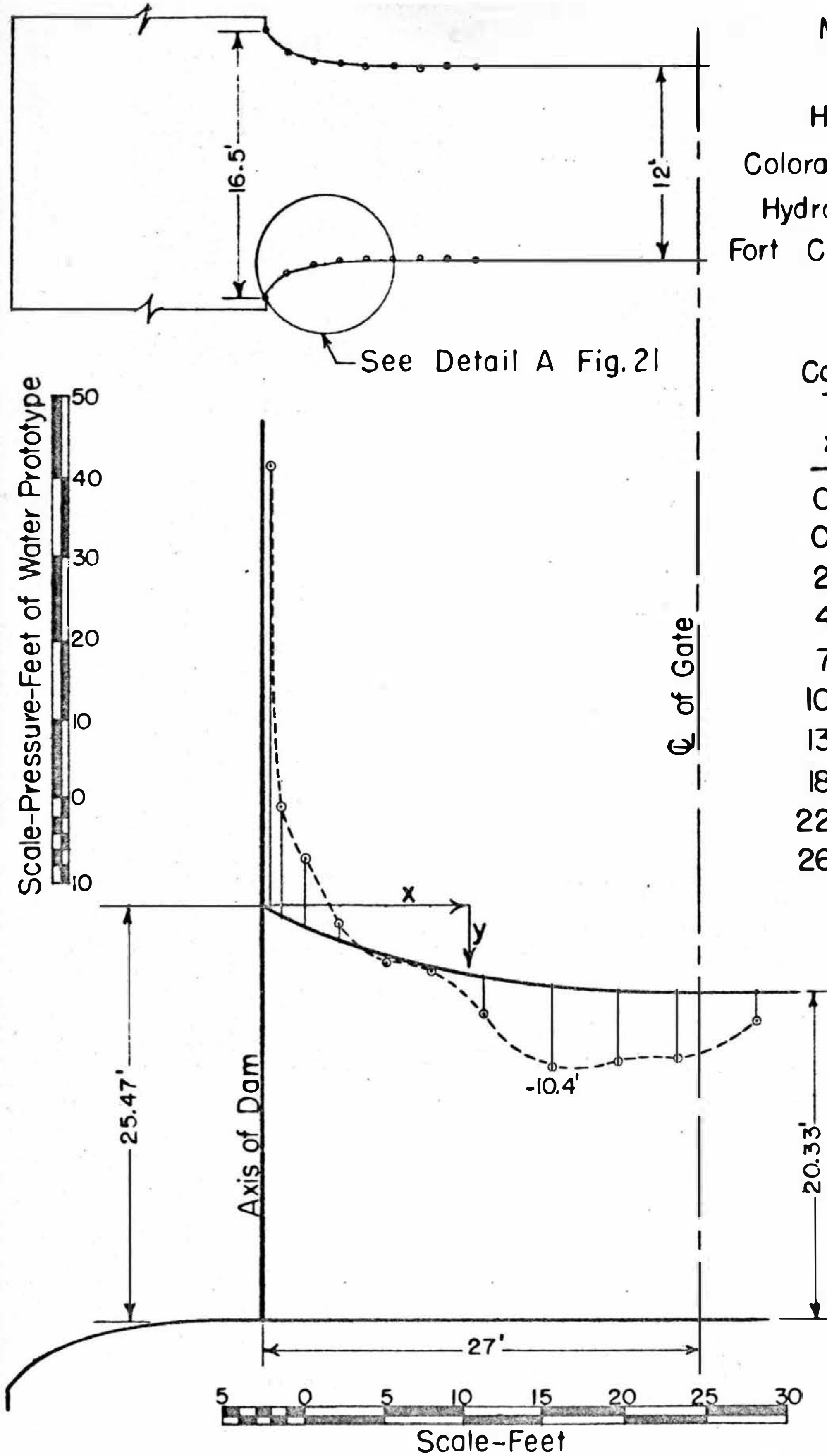


Coordinates of Top Curve

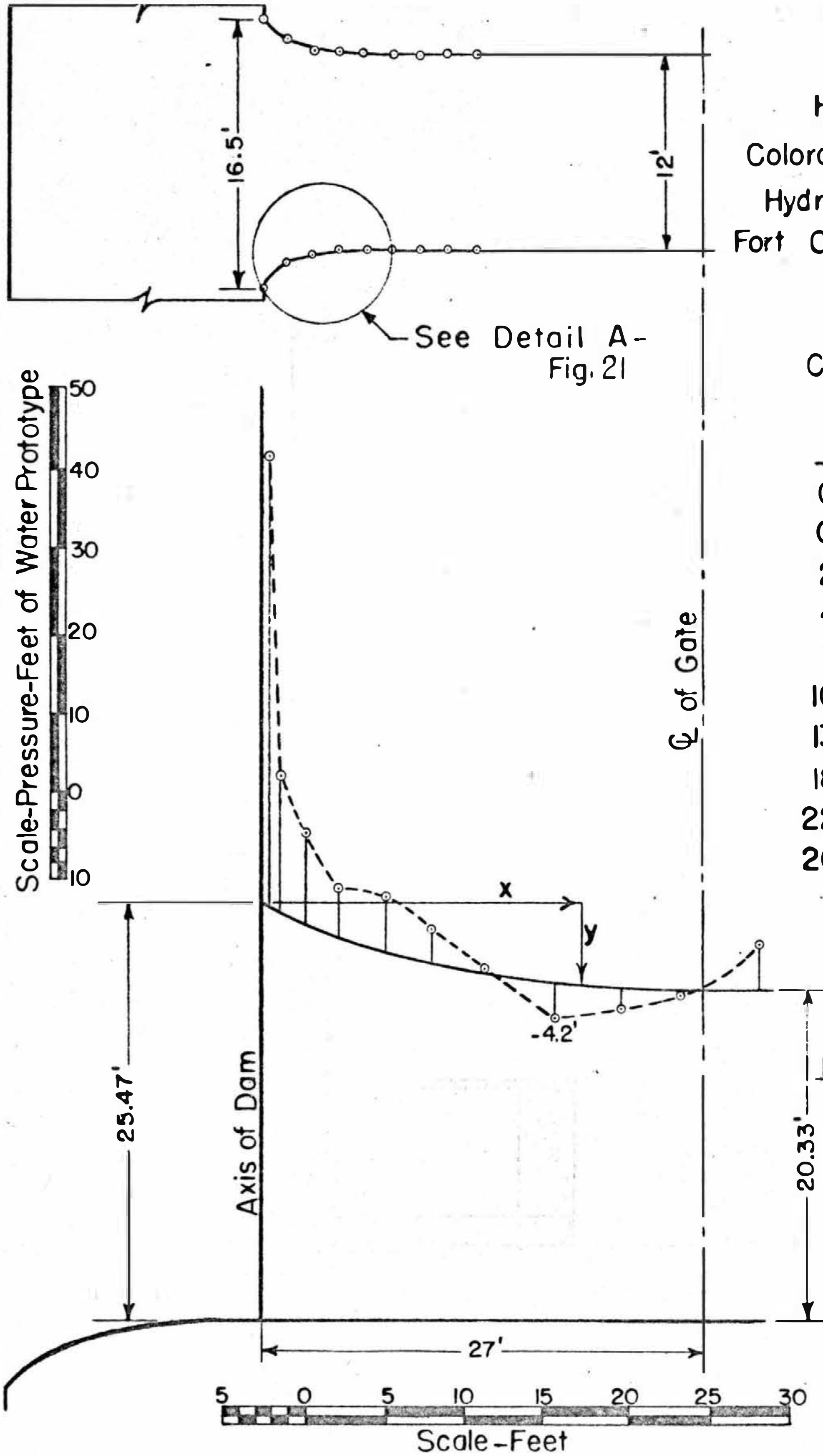
x	y
0.53'	0.30'
0.97'	0.53'
2.60'	1.27'
4.73'	2.03'
7.60'	2.93'
10.53'	3.63'
13.80'	4.20'
18.00'	4.77'
22.20'	5.07'
26.80'	5.14'

Note
H.W. El. 630.9

Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 4
1:40 Sluice Inlet Model

Fig. 25

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949



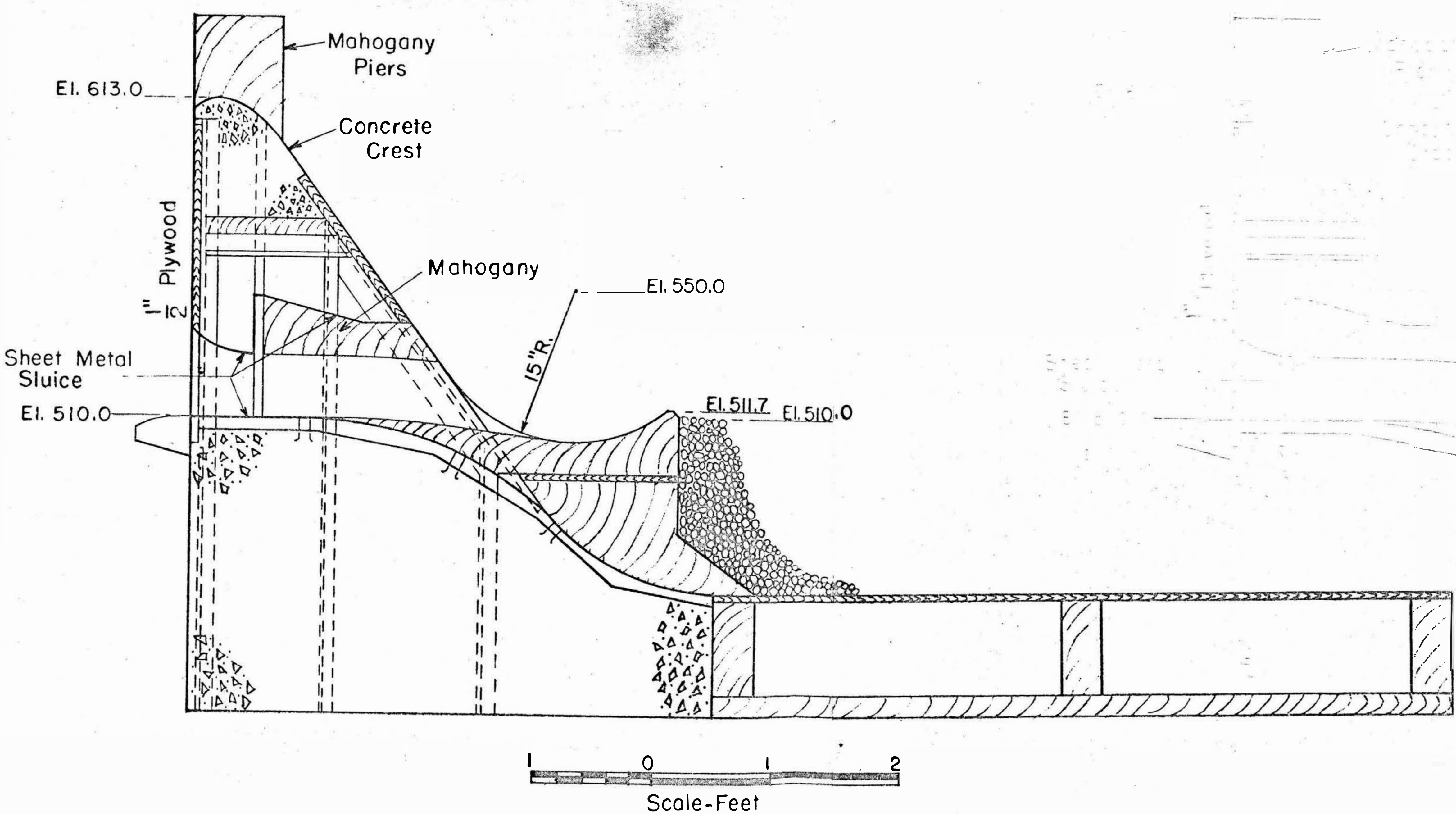
Coordinates of Top Curve

x	y
0.53'	0.30'
0.97'	0.53'
2.60'	1.27'
4.73'	2.03'
7.60'	2.93'
10.53'	3.63'
13.80'	4.20'
18.00'	4.77'
22.20'	5.07'
26.80'	5.14'

Note
H.W. El. 629.2
Outlet Contrated so that Pressure at the gate is raised 7.5 Ft.

Piezometer Location and Pressure Distribution
Sidewall Revision No. 2 - Roof Revision No. 4
Pressurized Sluice
1:40 Sluice Inlet Model

Fig. 26

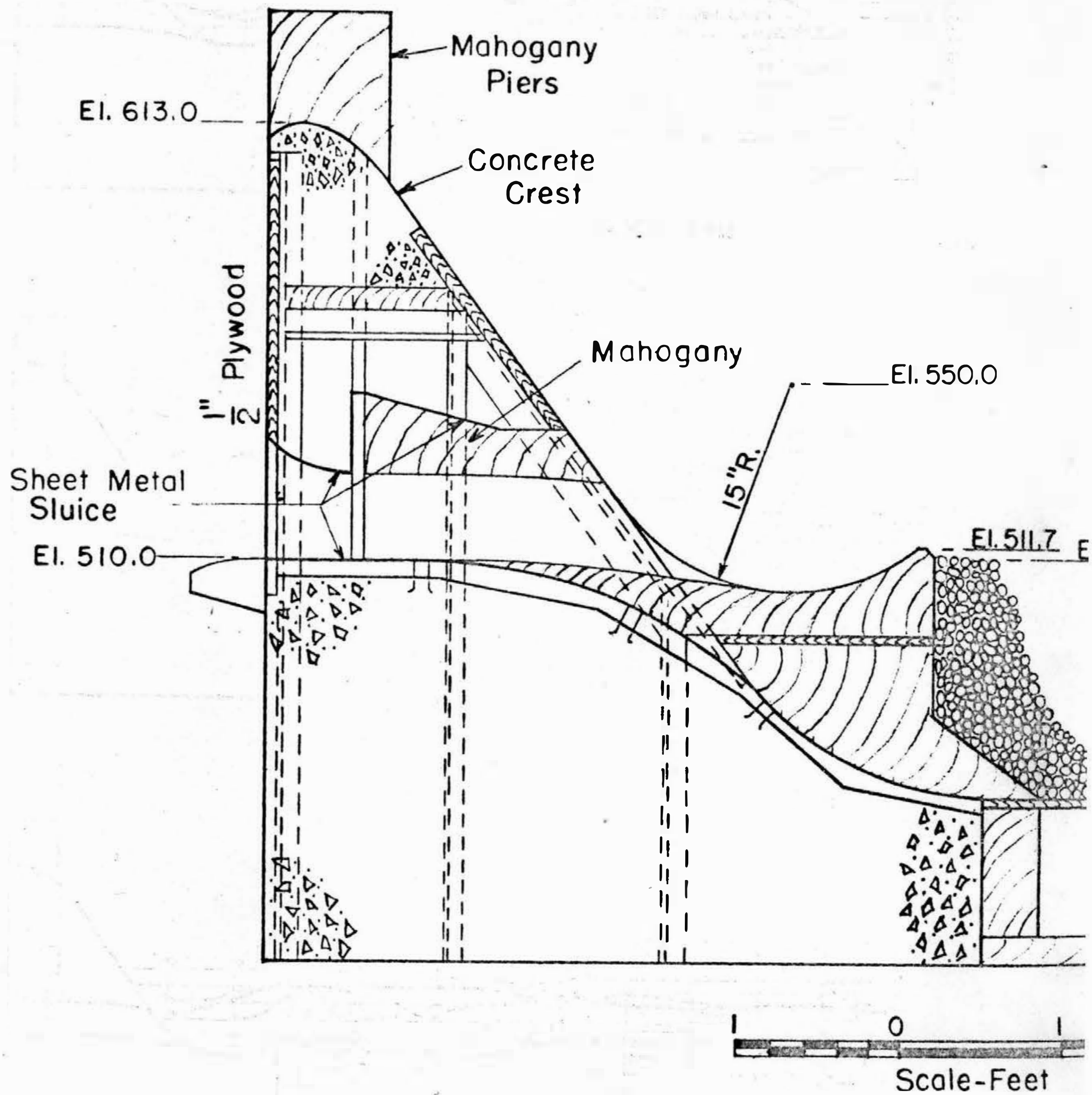


Elevation

1:40 Auxillary Spillway Model

Model Studies
of
HIRAKUD DAM

Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec. 1949

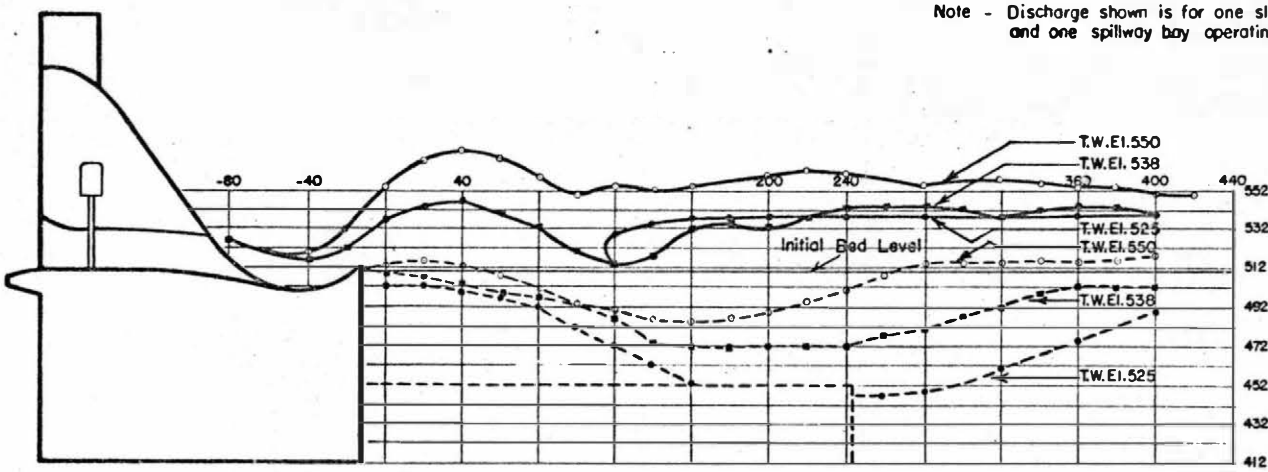


Elevation

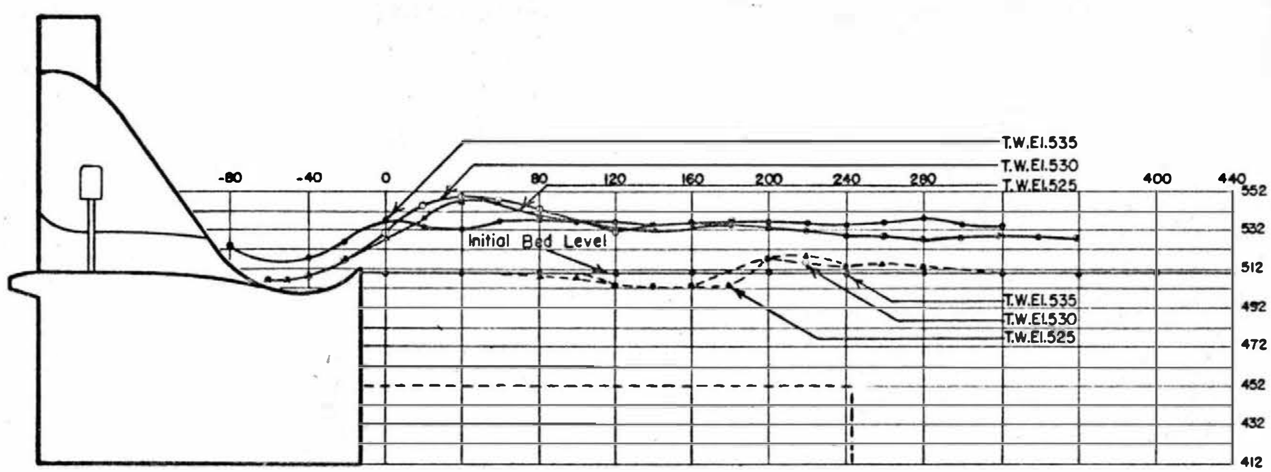
1:40 Auxillary Spillw

Fig. 28

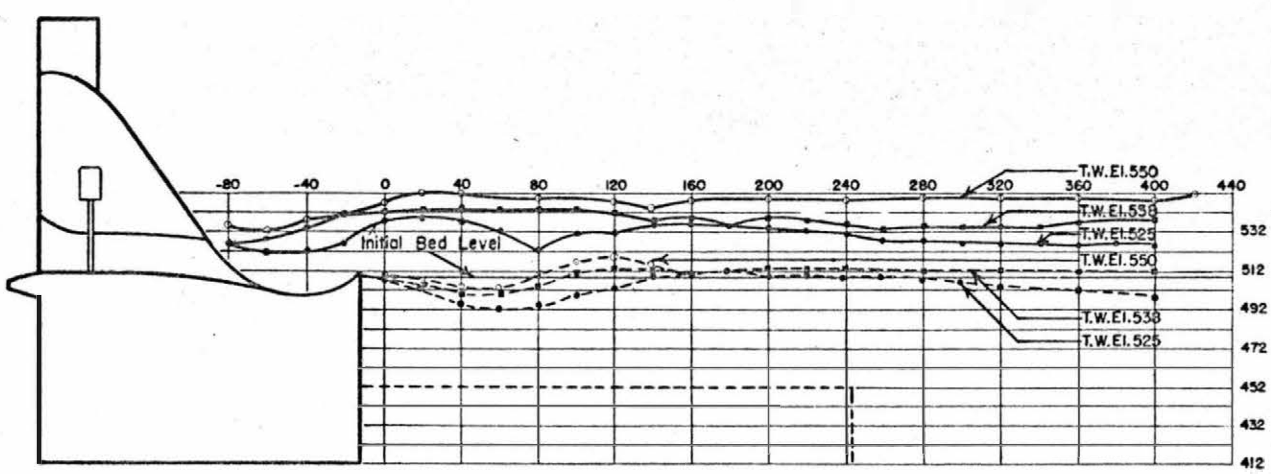
Note - Discharge shown is for one sluice and one spillway bay operating.



H.W. El. 630.2 Q = 22,770 c.f.s.



H.W. El. 630.4 Q = 5,400 c.f.s.
Crest Flow Only



H.W. El. 590.3 Q = 14,600 c.f.s.

Water Surface and Erosion Profiles
1:40 Auxiliary Spillway Model

Model Studies
of
HIRAKUD DAM
Colorado A&M College
Hydraulic Laboratory
Fort Collins Dec 1949

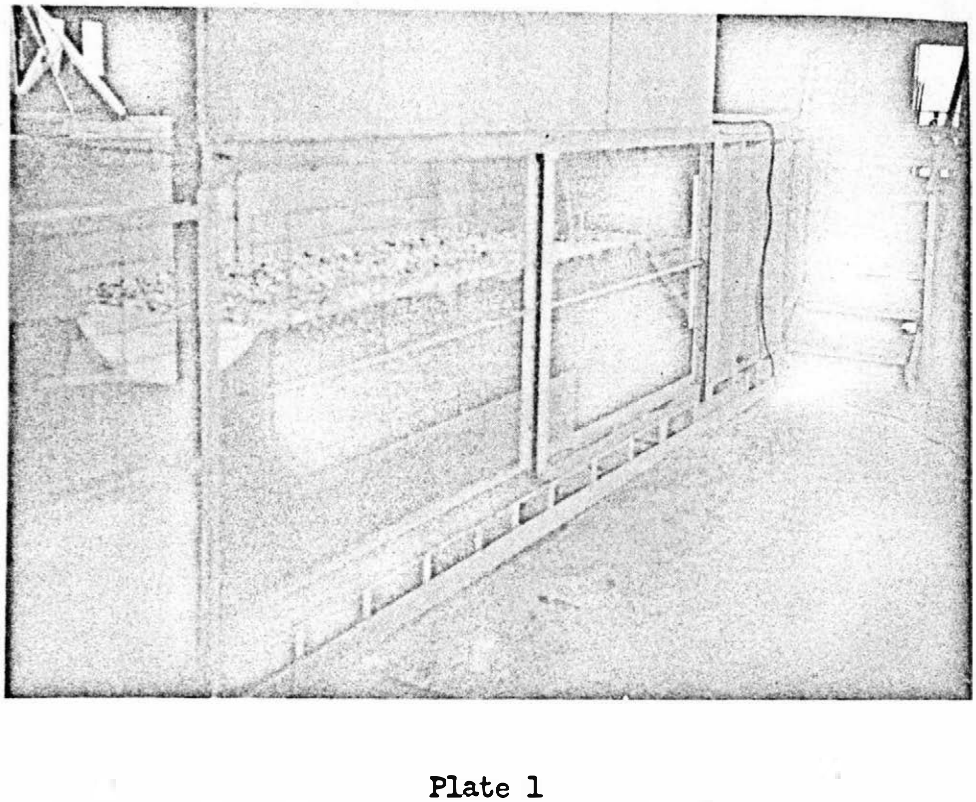


Plate 1
Glass-walled Channel - Sectional Model
River Bed Molded to Shape for Testing
1:40 Auxiliary Spillway Model

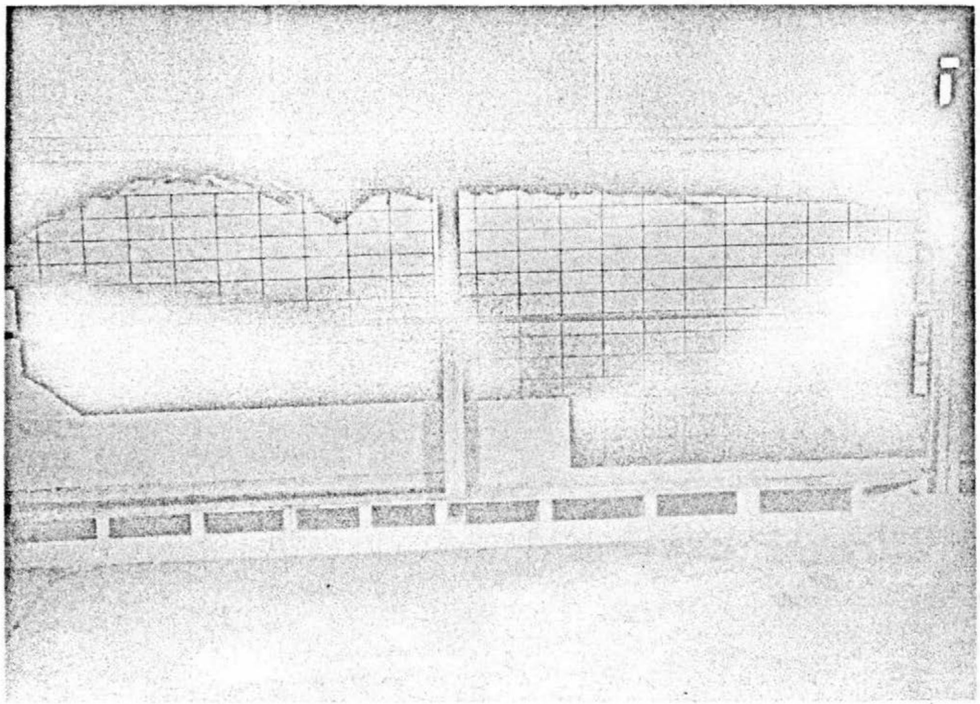


Plate 2
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 525, 20-min Erosion Test

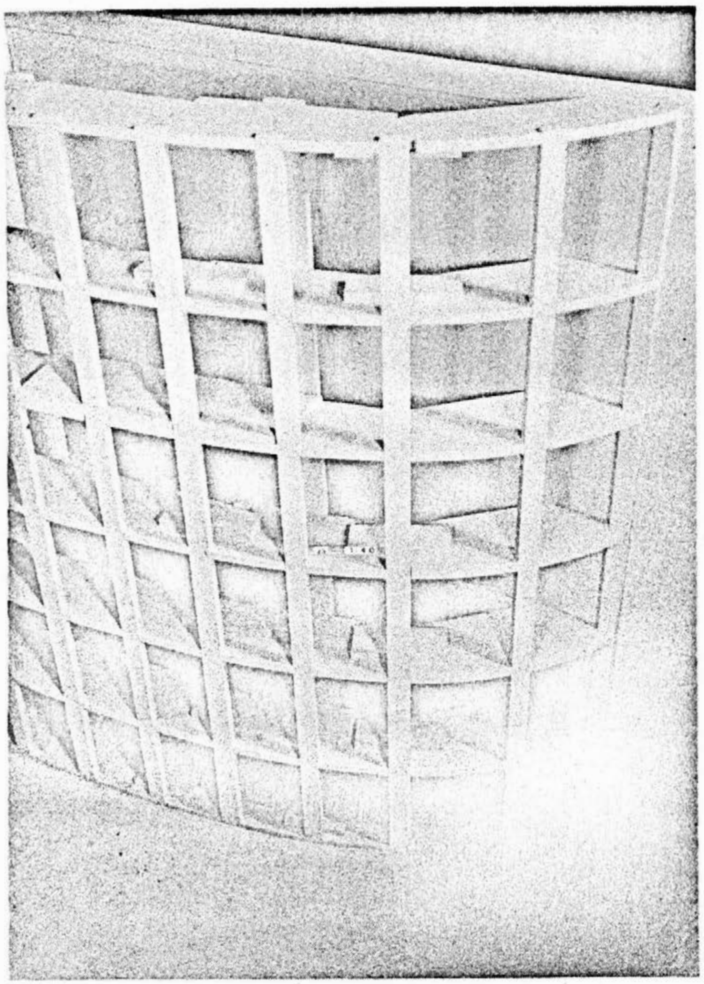


Plate 3
Head Box - Sectional Model
Transition Framework Ready for Plywood Covering

Plate 4
Head Box - Sectional Model
Partially Covered Transition Framework

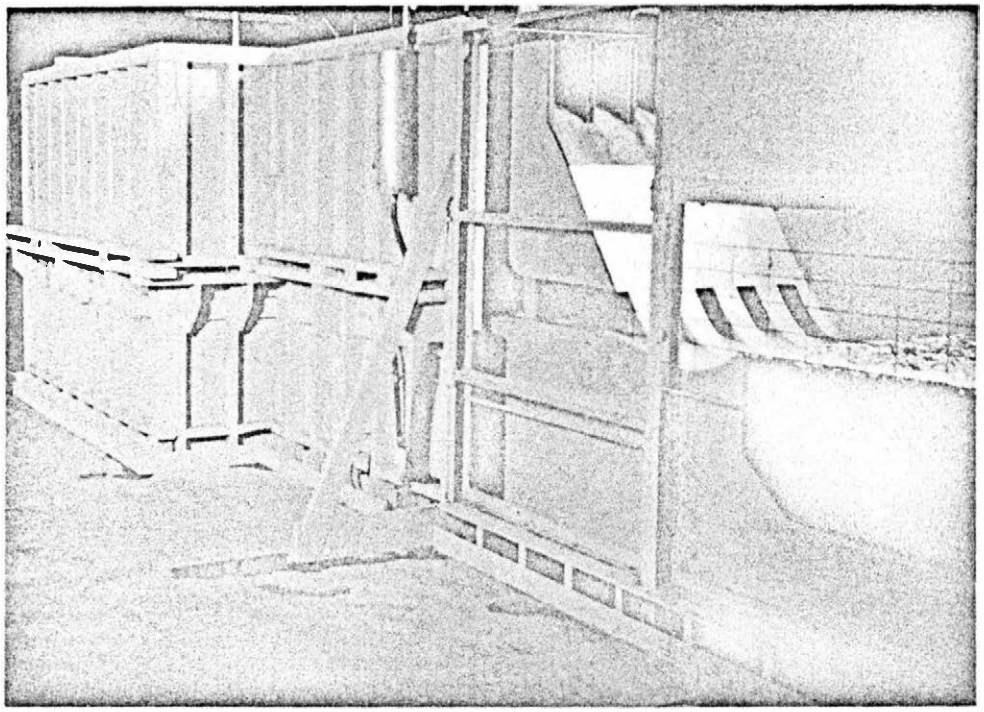


Plate 5
Head Box - Sectional Model
General View Showing Whalers, Hook Gage Well, and Spillway Structure

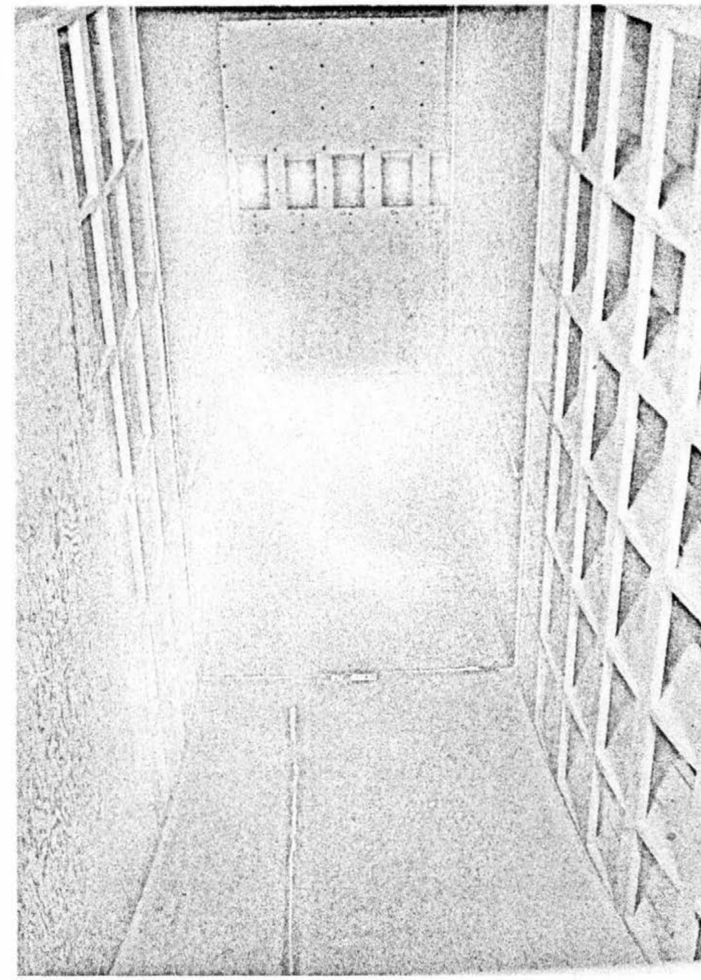


Plate 6
Head Box - Sectional Model
Partially Covered Transition Framework and Upstream Face of Dam

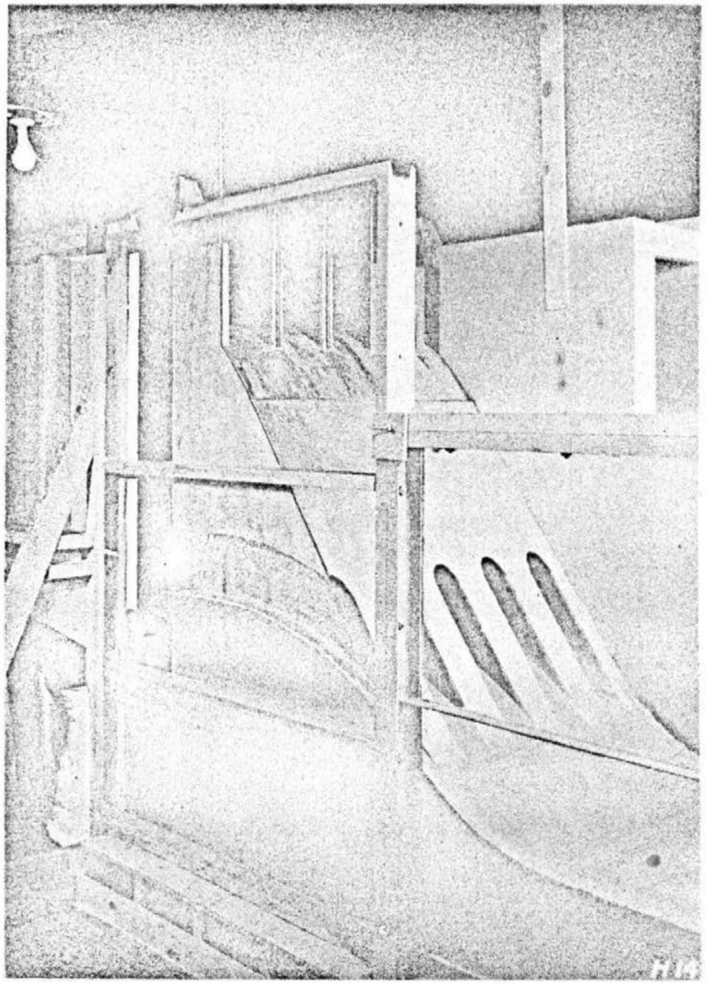


Plate 7
Spillway Structure - Sectional Model
Completed Assembly of Model

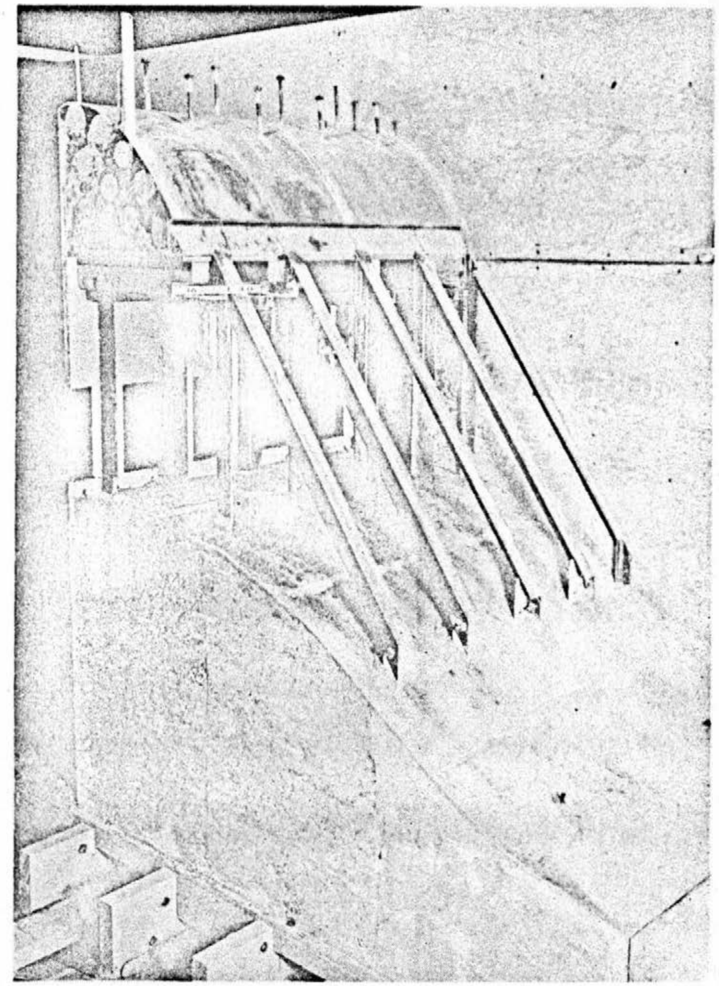


Plate 8
Spillway Structure - Sectional Model
Completed Concrete Base and Crest
Framework for Sluice Assembly

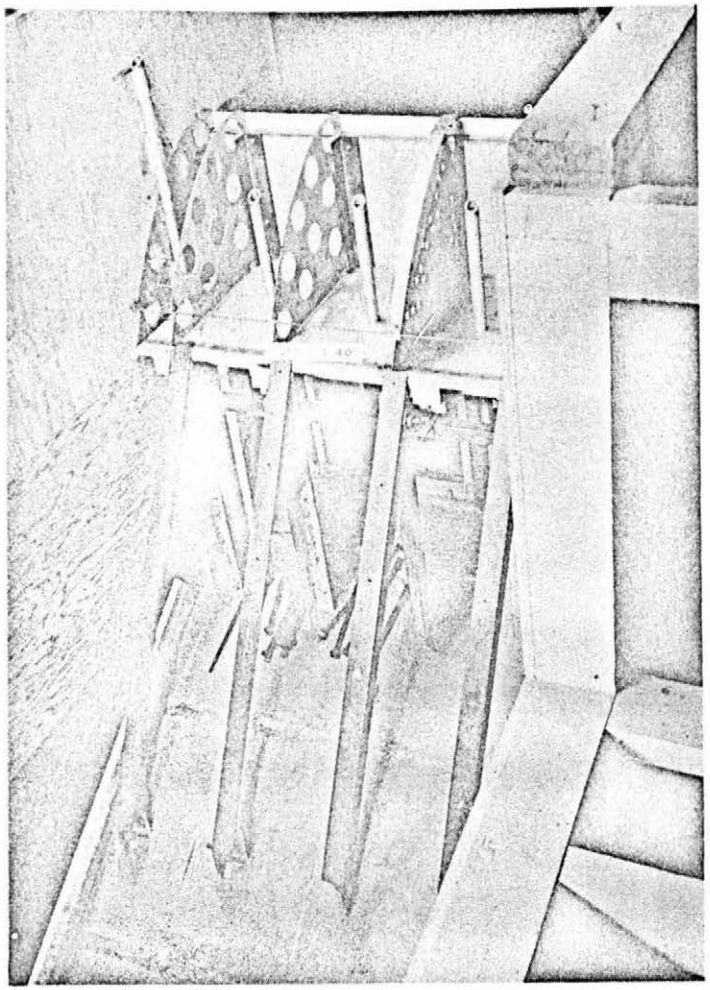


Plate 9
Spillway Structure - Sectional Model
Completed Concrete Base, Spillway Framework,
and Crest Assembly

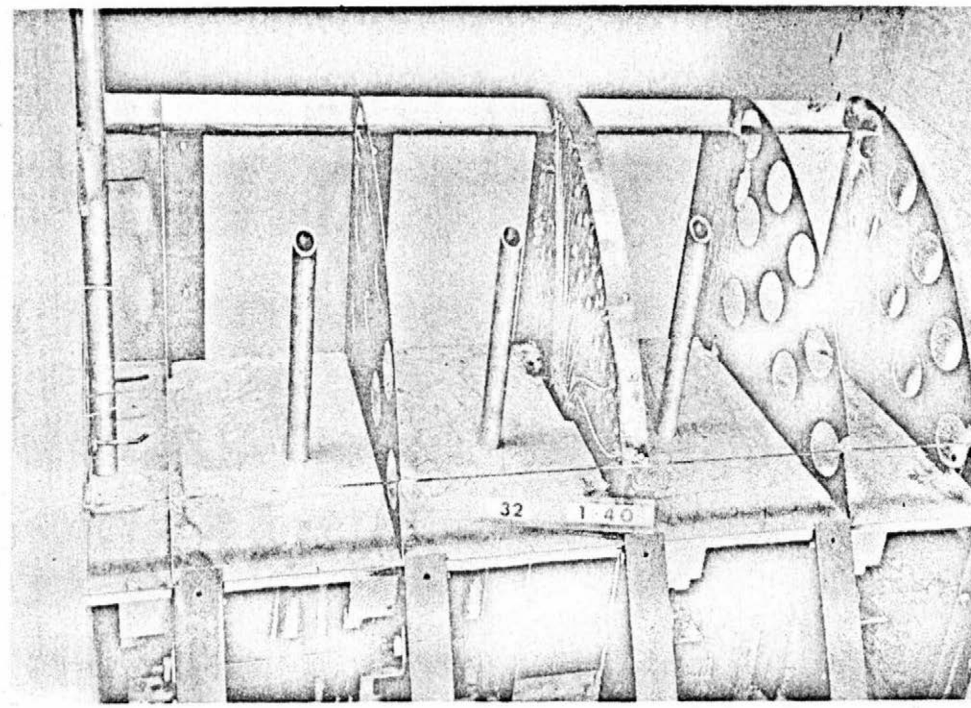


Plate 10
Spillway Structure - Sectional Model
Crest Assembly Showing Templets, Piezometer Section,
and Guide Tubes for Sluice Gate Control Arms

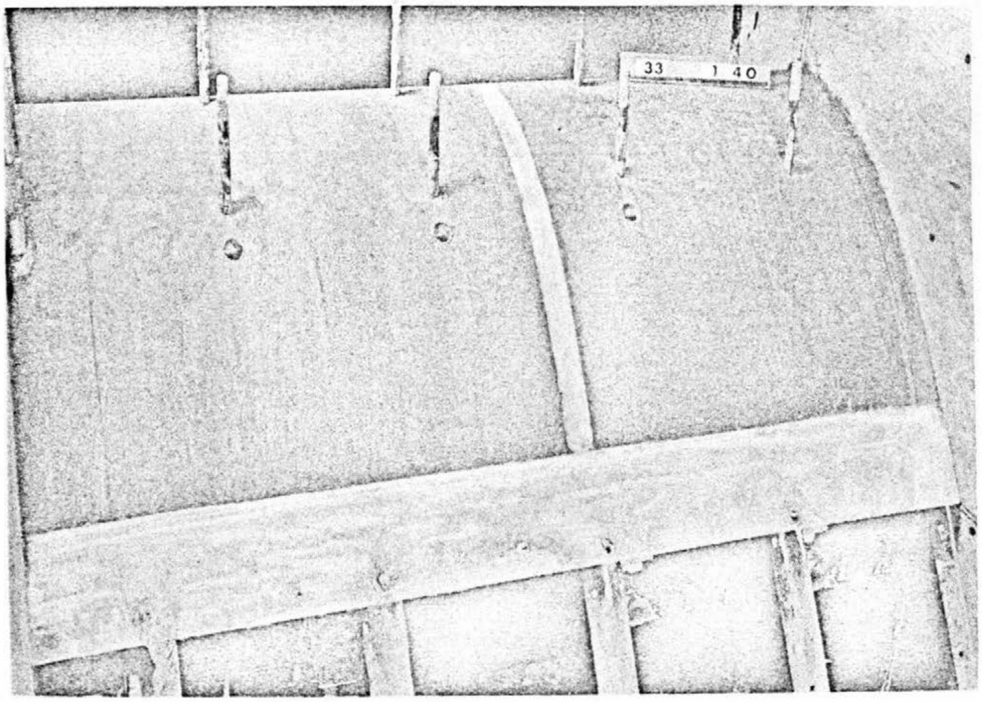


Plate 11
Spillway Structure - Sectional Model
Completed Crest Showing Pier Anchor Bolts,
Piezometer Section, and Guide Tubes for Sluice Gate Control Arms

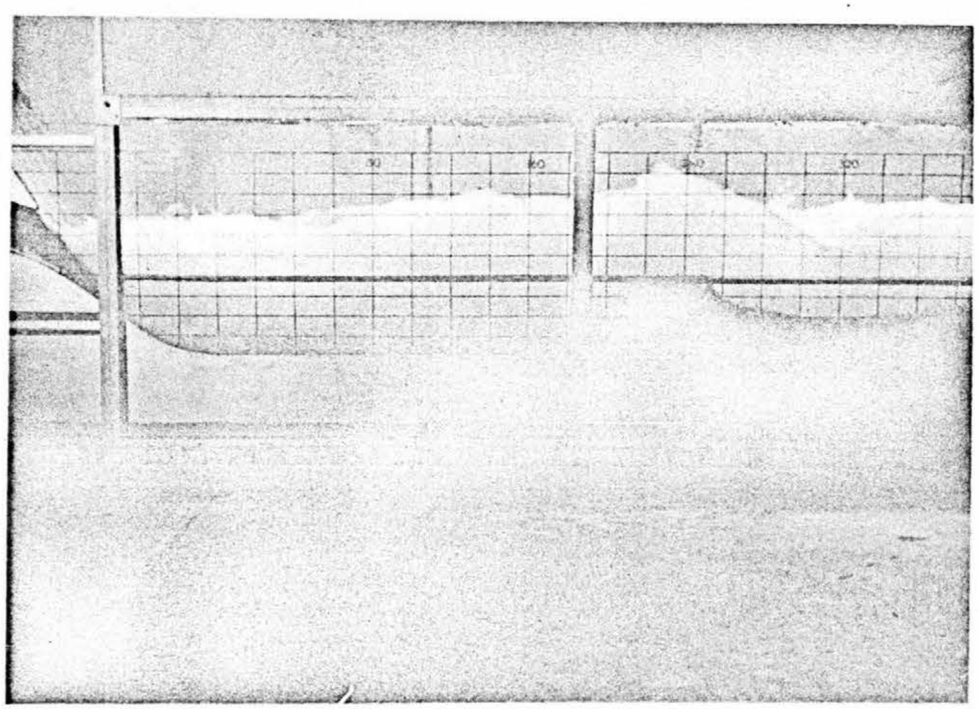


Plate 12
Water Surface and Erosion Profiles - Service Spillway
40-ft High Trapezoidal Sill 420 ft from Axis of Dam
HW 625, TW 520, Apron 452

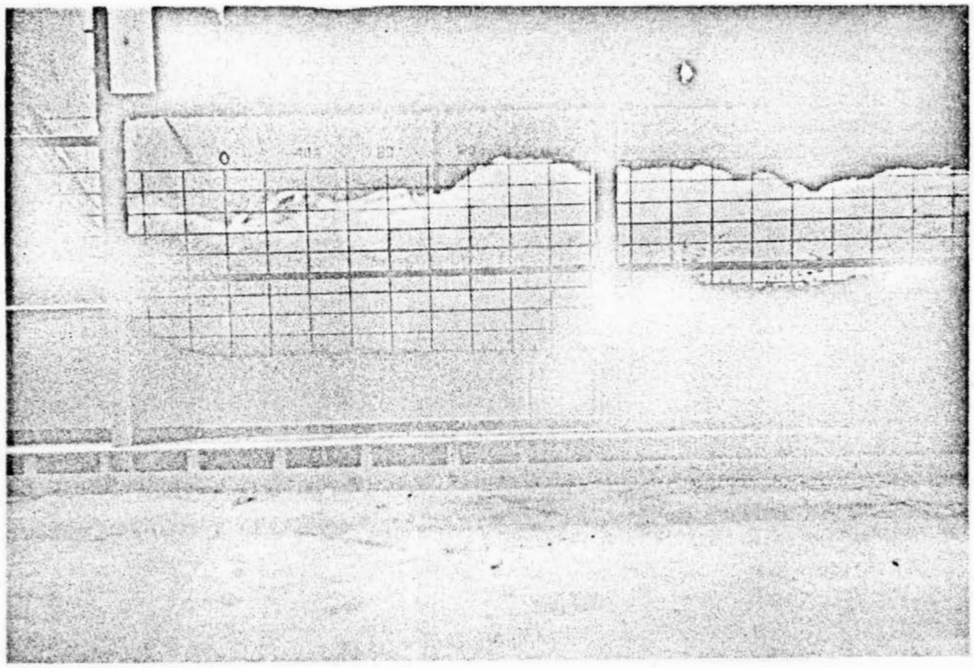


Plate 13
Water Surface and Erosion Profiles - Service Spillway
40-ft, 34° Dentated Sill 420 ft from Axis of Dam
HW 630, TW 520, Apron 452

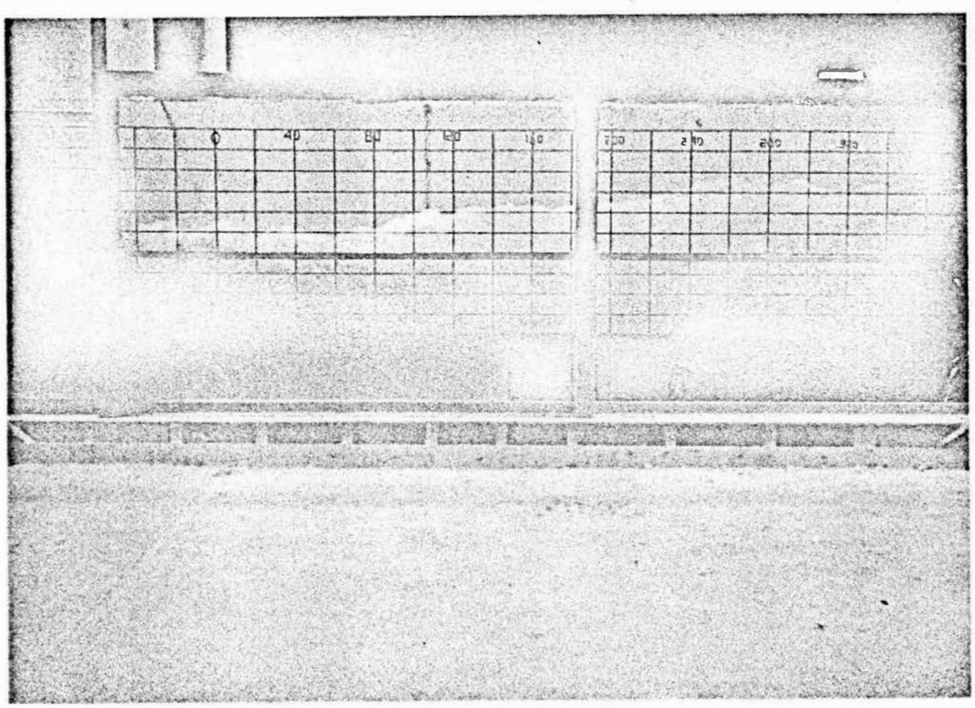


Plate 14
Water Surface and Erosion Profiles - Service Spillway
10-ft, Non-dentated Sill 420 ft from Axis of Dam
HW 630, TW 520, Apron 452

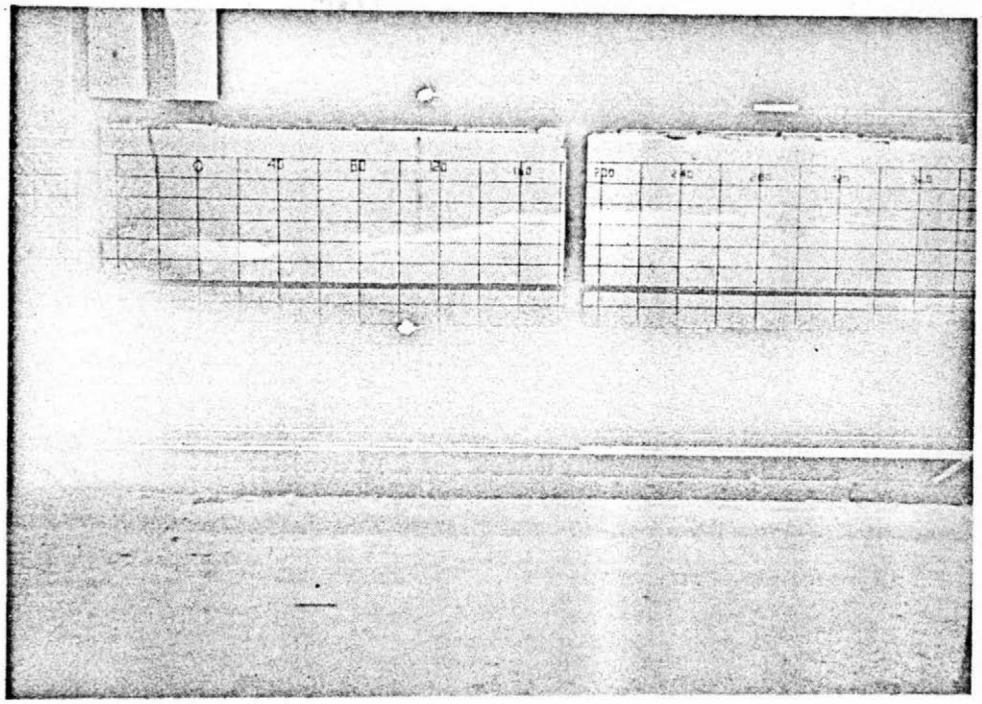


Plate 15
Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 400 ft from Axis of Dam
HW 630, TW 520, Apron 452

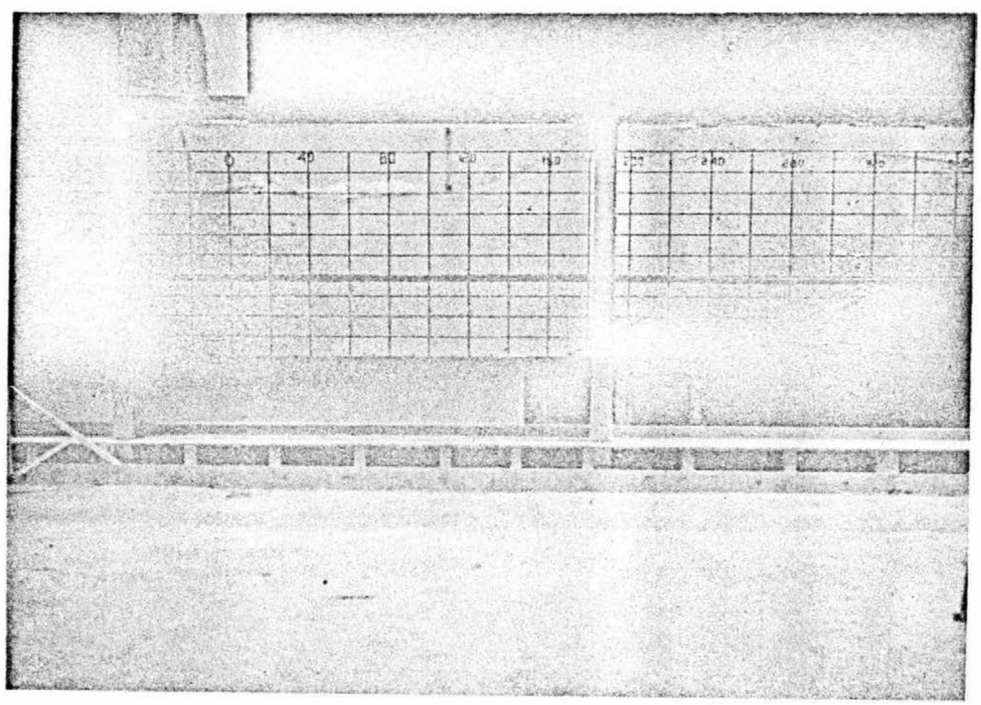


Plate 16
Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 400 ft from Axis of Dam
HW 630, TW 550, Apron 452

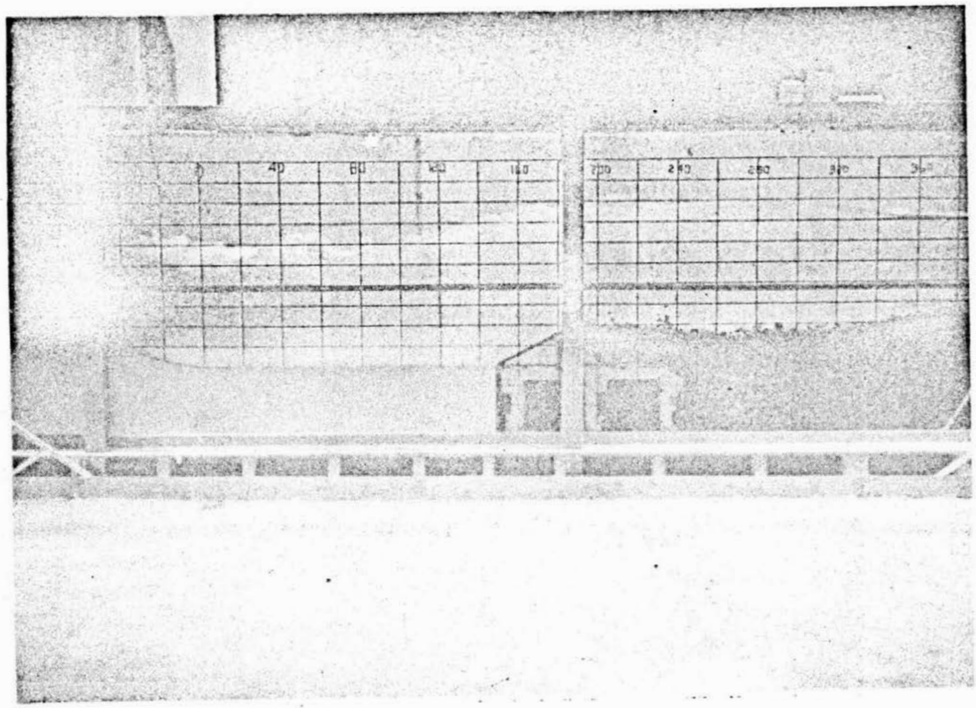


Plate 17
Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 380 ft from Axis of Dam
HW 630, TW 520, Apron 452

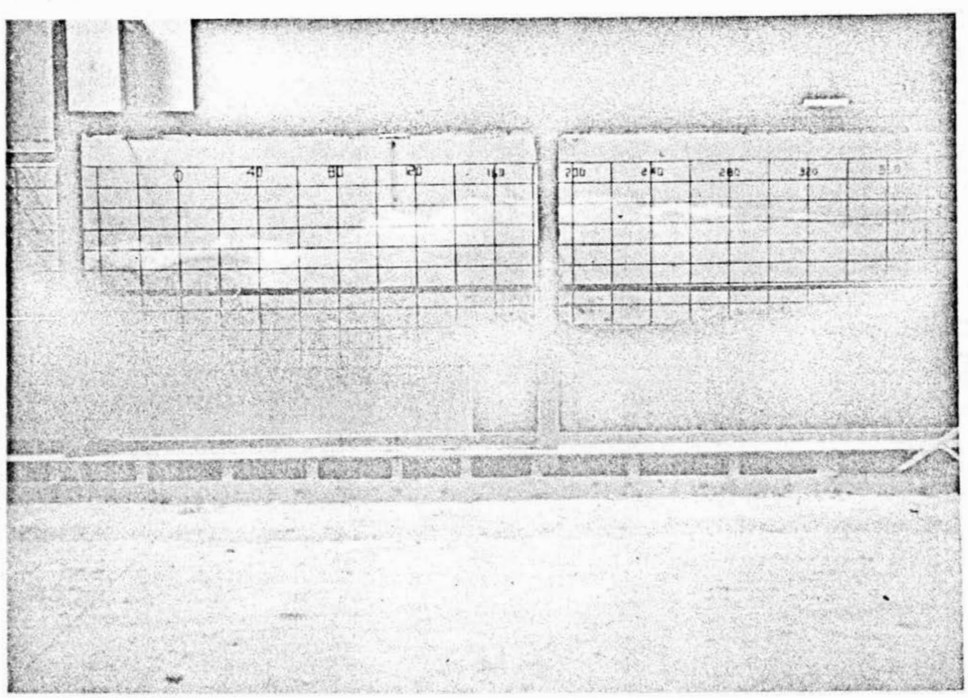


Plate 18
Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 336.7 ft from Axis of Dam
HW 630, TW 520, Apron 452

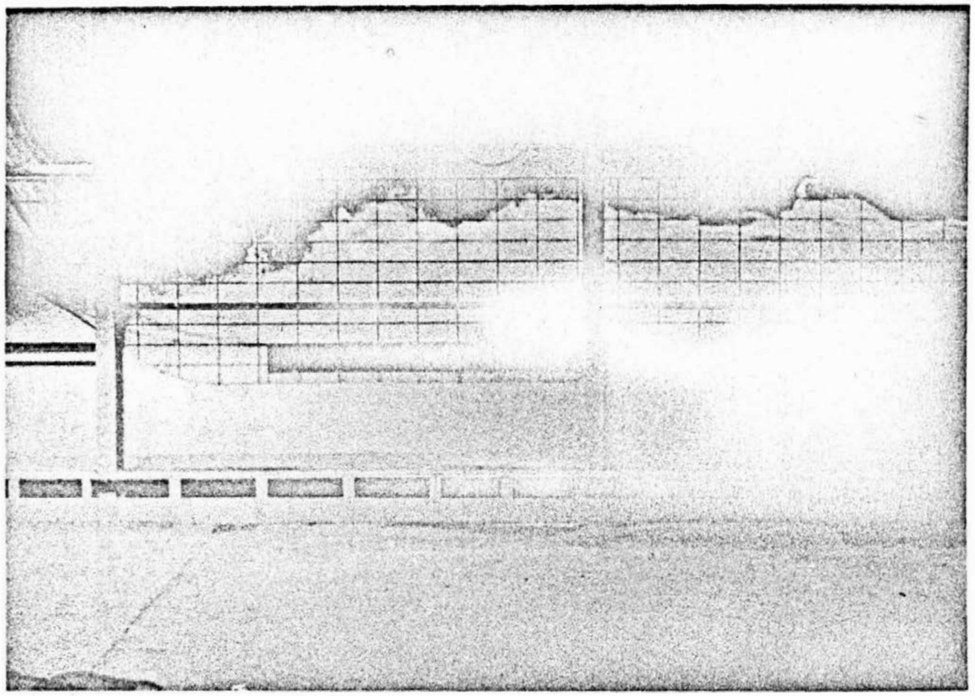


Plate 19
Water Surface and Erosion Profiles - Service Spillway
20-ft, 27° Dentated Sill 360 ft from Axis of Dam
HW 630, TW 525, Apron 472

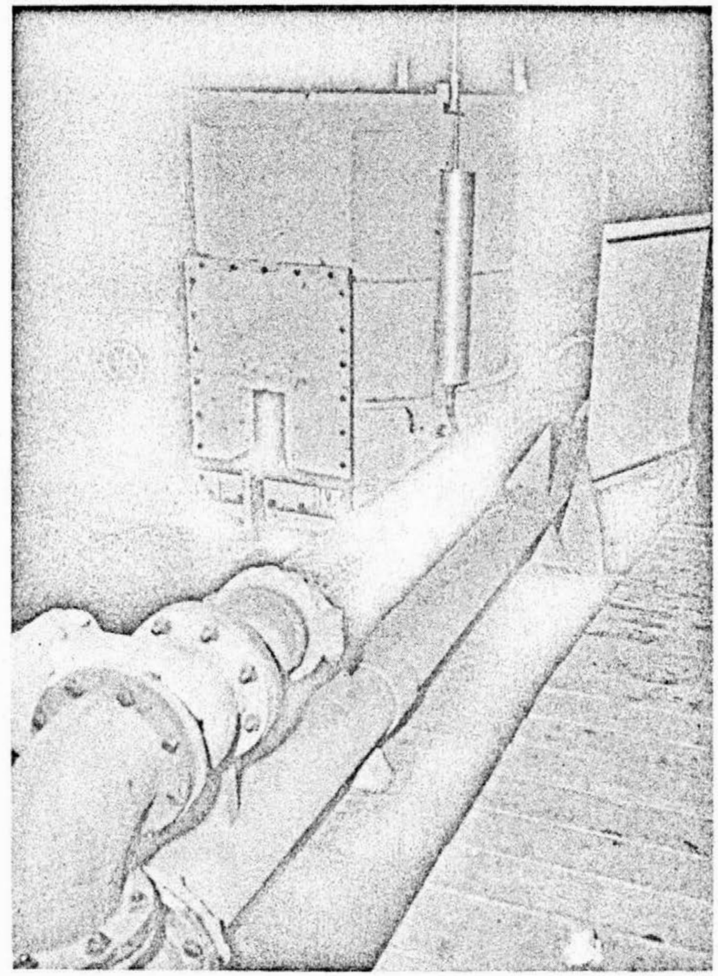


Plate 20
Sluice Inlet Model
General View Showing Water-supply Piping,
Entrance Tank, Orifice Plate, Sluice Floor,
Hook Gage Well, and Manometer Board

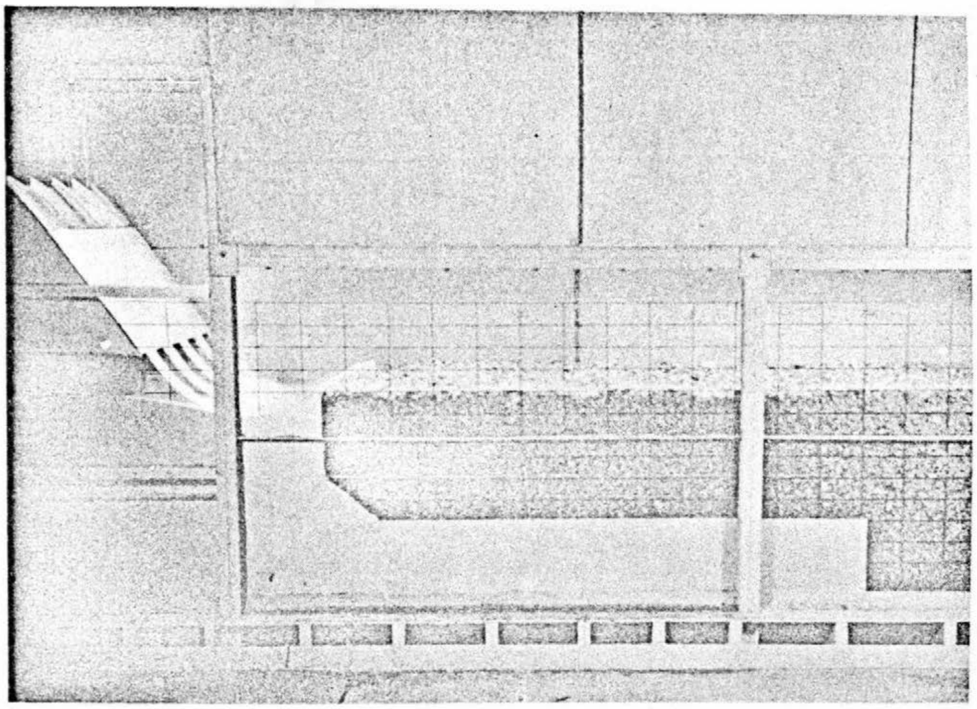


Plate 21
Glass-walled Channel - Sectional Model
Spillway Structure and River Bed
Prepared for Testing

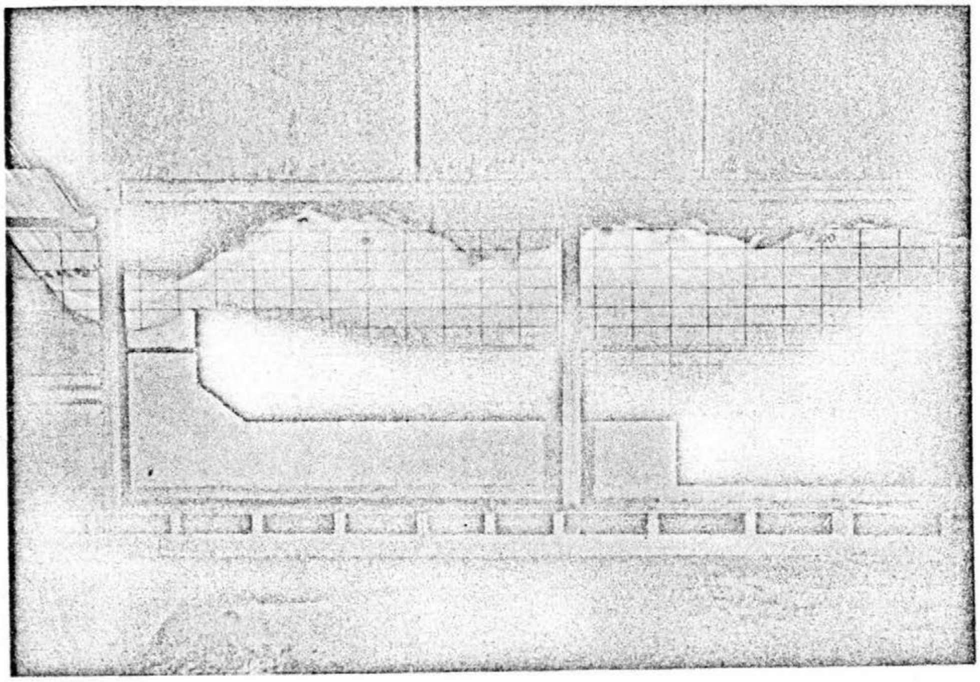


Plate 22
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 538, 20-min Erosion Test

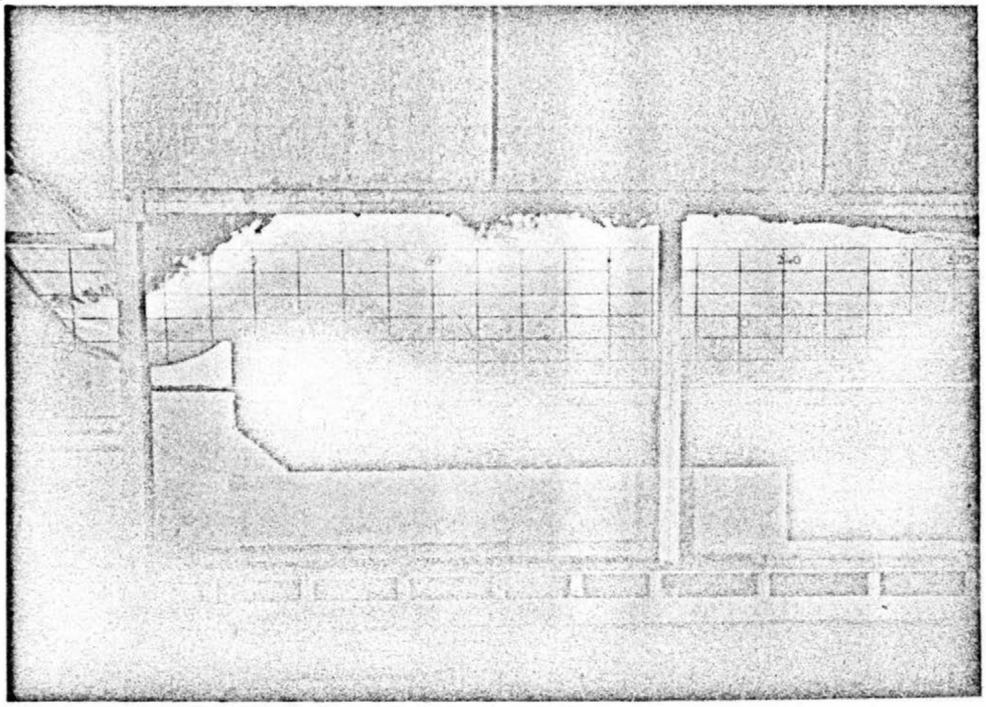


Plate 23
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 550, 20-min Erosion Test

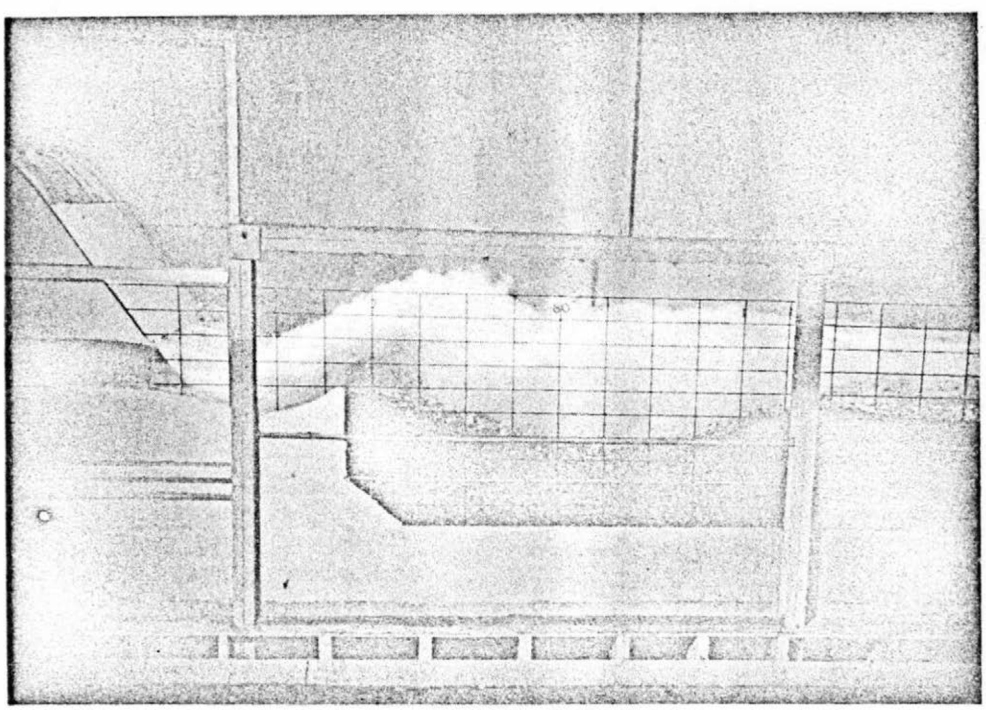


Plate 24
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 590, TW 525, 20-min Erosion Test

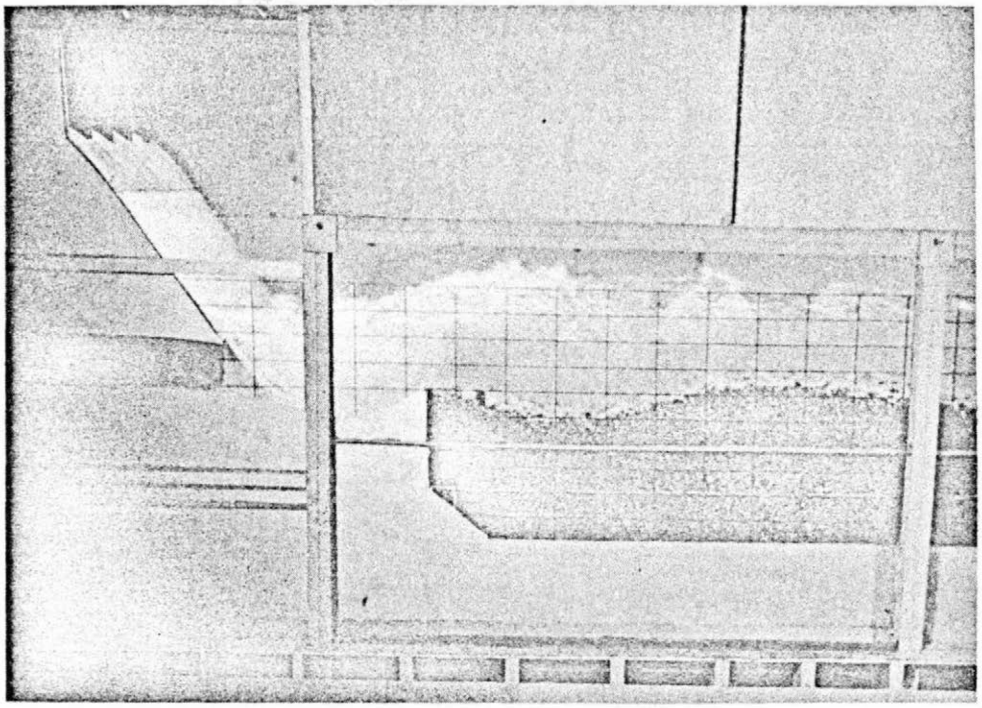


Plate 25
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 590, TW 550, 30-min Erosion Test

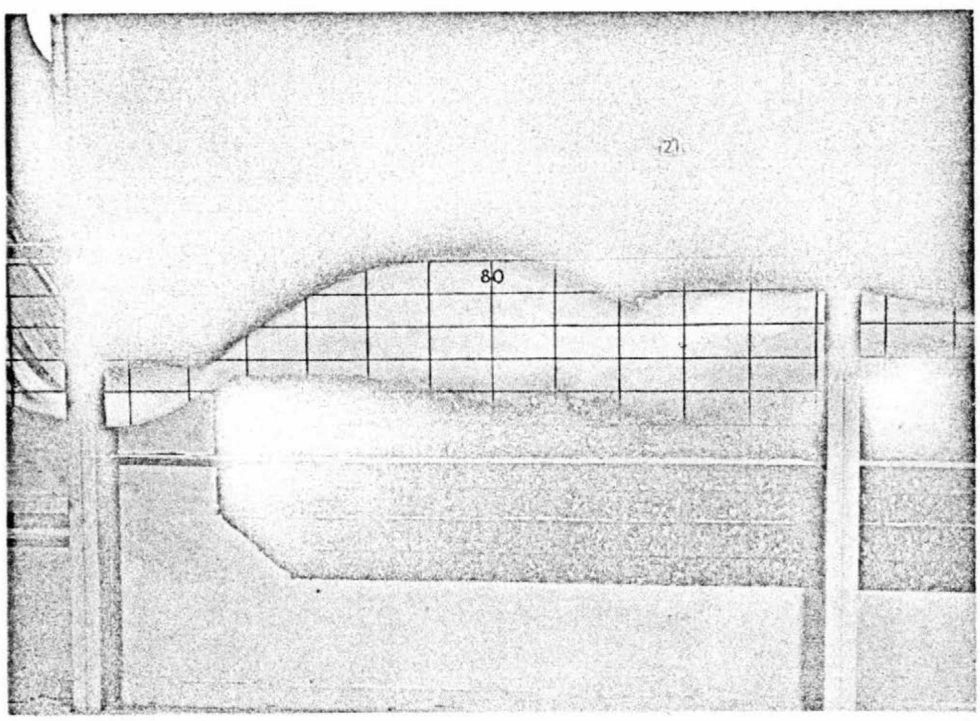


Plate 26
Water Surface and Erosion Profiles - Auxiliary Spillway
HW 630, TW 525, Crest Flow Only, 20-min Erosion Test