BOARD OF WATER ENGINEERS

THE STATE OF TEXAS

# A REPORT ON MODEL SPILLWAY STUDIES

Prepared under the direction of Ivan M. Stout,
Chief Engineer, State Board of Water Engineers,
in cooperation with Dr. Walter L. Moore,
The Department of Civil Engineering at The University of Texas.

Program Sponsored by the following Consulting
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September 1, 1954

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## INTRODUCTION

Pursuant to the objectives and procedures of the Cooperative Spillway Demonstration Project by the Board of Water Engineers and the Department of Civil Engineering at the University of Texas, the following report on model spillway studies is presented.

The Board has selected three examples of spillways of particular interest for model study. Under the supervision of Mr. Ivan M. Stout, Chief Engineer, models of each project were constructed and placed in the model basin provided by the Hydraulics Laboratory at the University of Texas. Tests to determine the hydraulic characteristics of each model were performed under the joint supervision of Mr. Stout and Dr. Walter L. Moore, Department of Civil Engineering at the University of Texas.

At the request of the Board, A. H. Woolverton, Consulting Engineer, Austin, Texas, has prepared the report on the Hicks and Pure Oil Company Projects. The report of the Kuhn model study was prepared by the staff of the Board.

# COMMENTS ON MODEL STUDIES FOR THE HICKS AND PURE OIL COMPANY DAMS AND SPILLWAYS

## GENERAL

The construction of a dam or other project for control of water generally involves large expense. In order to reduce the construction and maintenance to a minimum, it has been found that study and research are necessary. Such work requires adequate topographic work, preparation sometimes of many alternative plans, and study of the effect of the flow on the structure when relief facilities, such as spillways and outlet works, actually begin to operate.

The use of models of the spillways, channels, and other facilities, built on a small scale of from 30 to 1 to 120 to 1, is in general use by many organizations. It is believed the work on the models for the Hicks and Pure Oil projects has been excellent, and will be of great benefit to indicate desirable construction features for appurtenant spillways for impounding reservoirs, and how money to pay for such appurtenances can be best expended. Continued study of models of other structures will permit even more balanced design and construction and result in additional saving of funds.

## SPILLWAY OUTFLOW

The amount of the discharge of a spillway can generally be expressed by formula with reasonable accuracy for practical use. The action of the water in passing into the spillway inlet, over the crest, along the discharge channel below the crest, and in the area where the flow finally enters the stream channel below the dam, however, cannot be so accurately expressed, because of many uncertainties such as erosion possibility, foundation material, and the layout.

One way to gain information on the action and erosion along channels that transport large volumes or capacity of flow through a spillway is by observation of such outflow when it occurs. Large outflows are caused generally by storm rainfall of more than normal amount. Where the spillway design permits a discharge of the amount compatibable with the yield from a large storm run off, relatively large spillway outflows may be rarely expected, and few engineers are prepared to observe the operation at that time, although the results of the action during smaller floods may be sometimes observed, as the smaller flows are more frequent.

The topography rarely will permit the same design for a spillway for a dam in all its aspects. Therefore, if the action at one particular dam is inspected it is probable that a different result would occur at another dam, because of even small differences at the inlet, crest, or along the channel of the spillway. It has, therefore, been found beneficial in obtaining best designs, to make a thorough study of the conditions for each separate project. Where the uncertainties encountered indicate that safety of the structure or high construction costs will be involved, the expense of a model study is considered minor as compared with a possibility of reducing or preventing excessive construction costs.

Many spillways, except for the large dams, are designed without the use of a model. In many instances the design is successful, but in many cases, where bold designs are utilized, the use of the spillway when necessary results in costly maintenance and repairs. Sometimes, funds are not available for the necessary repair, and erosion and deterioration of the spillway continues, and may eventually result in loss of the dam, the stored water, and the money already invested. Considerable study and tests are necessary for curves that are located near the toe of the dam, or near structures that might be undermined, or for similar conditions.

## MODELS

Except for large laboratories that have operated for a considerable period, and have sufficient funds for the purpose, it is generally necessary to construct all the channels of a hard setting material, such as plaster of paris. Cement grout is sometimes used. The scouring effect on earth and sand lined channels cannot be very well observed in such models, but the direction of the flow, cutting possibilities, and similar action, can be observed. Experience in design of structures has caused the writer to believe that honest study of this type of model is beneficial, and will lead to a more balanced design, and lower over all costs.

When a model is built, it should represent the actual condition that will exist, from a reasonable distance upstream of the inlet to a point well downstream of the outlet end. Small changes made for convenience may have considerable effect on the results obtained. Concrete lining if well designed has high resistance to erosion. An unlined channel, especially if located in sand, has a very low resistance. To obtain results that show the actual conditions the channels on the model should be lined with material that will afford the same relative resistance to flow in the model that would exist in the completed structures and channels during the operation of the spillway. It is almost impossible to obtain a relatively correct reproduction of the materials, but the indication of the action during study of the model flow is very helpful in design, and in obtaining information to prevent high construction costs.

The flow through a model may be varied to permit the conditions caused by a portion of the maximum flow to be observed. It may be indicated that the maximum outflow will cause no damage, but that flows of one-quarter or one-half of the maximum flow will cause damage. Such information may prevent failure of a structure by erosion from small spillway discharges.

## SAFE CHANNEL VELOCITY

The design of a spillway channel should provide for velocities that will not scour or erode the lining or foundation material. Channels for canals are designed generally on very flat grades to cause low velocities. Channels for spillways must generally have steep grades. For channels that are not too long, the range of the velocity can generally be estimated by the following formula:

$$V = 8\sqrt{0.8\ h}$$

where "V" represents the velocity in feet per second, and "h" is the drop from the water surface in the lake to the point where the velocity is to be calculated. If the drop is about 50 feet, therefore, the velocity would be about 50 feet per second. Such a velocity would erode almost any unlined channel, or even lined channels, unless the lining should be of concrete or similar type material. It has been proposed in some cases that the flow could be spread to increase the friction to cause low velocities throughout the length of the channel, and along the steep descending grade. It is not proven that low velocities could be obtained even with excellent maintenance of the channel, as small local concentrations of the flow would quickly permit scouring velocities to exist. Where there are possibilities that the construction may be endangered by the operation of the spillway, and the conditions that will exist are not entirely clear, the use of a model of the spillway and appurtenant channels will at least indicate the places along the channel where dangerous conditions or velocities might develop.

A considerable amount of study and tests have been made to determine the velocities that are considered safe for different types of material. In general, the safe velocities have been found to be about as follows:

| Material | Safe Velocity Feet per second |
|---|---|
| Fine sand | 1.50 to 2.50 |
| Sandy loam | 1.75 to 2.50 |
| Fine gravel | 2.50 to 5.00 |
| Coarse gravel | 4.00 to 6.50 |
| Cobbles | 5.00 to 6.50 |
| Shales and hardpans | 5.00 to 6.00 |

## THE HICKS PROJECT

The Hicks project is located above or upstream of Lake Bridgeport on a tributary of Howard Creek in the watershed of the West Fork of the Trinity River, in the Charles Hartman Survey in Jack County. The location is about 13 miles northeast of Jacksboro. It is constructed of wetted and rolled embankment, to a height above the flow line of the stream of about 31 feet. The slopes of the dam and the spillway channel are sodded with Bermuda grass. The surface area of the lake at spillway level is about 63 acres, and the capacity of the lake at that elevation will be about 607 acre-feet. The drainage area is about 950 acres.

The dam was construct ed principally for flood control purposes, although the permit (No. 1579) granted to J. L. Hicks grants a permanent storage of 136 acre-feet, which volume may be utilized for silt retardation. That amount of water will cause a depth above the dam of about 14 feet. Assuming that the lower 136 acre-feet of storage will be full at the beginning of a maximum flood, a run-off of about 6 inches would be required to raise the lake surface to spillway level, which might occur from a rainfall of about 18 inches in 72 hours, under conditions favorable to run-off. Additional rain causing flow over the spillway for about 24 hours, with a maximum depth of about two feet would cause the discharge of about 400 acre-feet, and about 150 acre-feet would be required for the temporary storage above the spillway level. Use of the spillway in that manner, therefore, would require a run-off of about 1,000 acre-feet, which amount to a run-off of about 13 inches. A maximum flood in that range may sometime be expected.

The economic value of the structure for storage of flood water is important locally. It will provide almost complete flood control for all storm flows except those of such magnitude as to be classed as maximum possible floods with a run-off of more than 6 inches on the watershed. A maximum flood of that type may not occur in our life-time. The fact that it will occur at sometime, however, appears certain because of the great amount of information compiled concerning floods of that size. The time may be near, or distant. When it does occur, the successful operation of the spillway will be of great importance in preserving the investment, and for the protection of lives and property downstream of the dam.

## MODEL

A model of the dam and spillway was built on a scale of 100 to 1. That is, the bottom width of the spillway channel on the model, as an example, is 0.70 feet, while the width for the constructed channel is 70 feet. The decision to build a model for this particular structure was caused by the use of a spillway channel curved in plan and a layout such that the lower portions of the channel would lie adjacent to the downstream toe of the dam. It is generally believed that such a layout will cause the toe to be undermined during the operation of the spillway, unless adequate protection of the channel is provided. The design contemplated that the Bermuda grass sod in the spillway would permit safe operation during a maximum flood condition.

## DRAWINGS AND PHOTOGRAPHS ATTACHED

The following drawings and photographs are attached hereto:

| | |
|---|---|
| Drawing No. 1. | J. L. Hicks Project |
| Drawing No. 2. | Plan of embankment and spillway |
| Drawing No. 3. | Profile of center-line of spillway |
| Drawing No. 4. | Cross-section of spillway channel flow on model of structure |
| Drawing No. 5. | Section with levee along right bank |
| Drawing No. 6. | Section with superelevation on curve |
| Drawing No. 7. | With superelevation (continuation of section shown on Drawing No. 6.) |
| Drawing No. 8. | Section without superelevation and with altered entrance |
| Drawing No. 9. | Proposed changes in present service spillway of Rhines Reservoir |
| Photograph I. | Model as originally built. |
| Photographs IIa. and IIb. | Model with outer levee to contain spill |

| | |
|---|---|
| Photographs IIIa, IIIb, and IIIc. | Model with outer levee, superelevation and slightly improved entrance |
| Photographs IVa, IVb, IVc, IVd, IVe, and IVf. | Model with outer levee, no superelevation but greatly improved entrance |
| <u>Subtitle</u> | |
| IVa. | Die pattern shows bottom currents |
| IVb. | Die pattern shows bottom currents |
| IVc. | Drops of die show surface current |
| IVd. | Drops of die show surface current |
| IVe. | Dark spot die shows area where spillway is nearly dry. |
| IVf. | Close-up of improved entrance |
| Photographs Va and Vb. | Model without levee or superelevation but with improved entrance |
| <u>Subtitle</u> | |
| Va. | Die current shows bottom current |
| Vb. | Drops of die show surface current. |

## MODEL OPERATION

The model was operated for various amounts of spillway discharge. For very low flows, representing depths of about 6 inches above the actual spillway crest, the results were considered indeterminate. For more flow, the stream in the spillway began to indicate a pattern, and a tendency to increase in depth on the outside of the curves, and to recede in depth on the inside of the curves. Surface waves began to develop, which were reflected from the outside of the upstream curve, to and against the slope of the dam, and back across the channel to the outside slope of the channel at the downstream end. Velocities in the channel were not measured, but as friction was not excessive in the smooth channel on the model, the following approximate amounts would probably have existed on the actual structure:

| Estimated velocities for depth on crest as indicated | | | |
|---|---|---|---|
| Depth on crest | 0-6" | 2'-0" | 5'-0" |
| Discharge c.f.s. | 75 | 590 | 2,340 |
| Velocity in feet per second | | | |
| Station 1+ 90 | 3.2 | 6.3 | 10.0 |
| Station 3 + 25 | 16.8 | 17.2 | 19.0 |
| Station 6 + 55 | 31.5 | 32.0 | 32.4 |

The amount of flow was measured by a Venturi meter.

The model indicated conditions for scour would exist, but did not show scour because of its construction of hard setting material. It could not show the value of Bermuda grass sod to resist erosion. Most designs do not permit computed velocities of more than 7 or 8 feet per second, because that experience and operation have shown that cutting and scouring will begin at about that point. Bermuda grass and other grasses will tend to mat where the growth is heavy and line the channel, which will afford protection. If such lining were perfect, rather high velocities might obtain without damage to the channel. Such linings are never perfect, however, and increased velocities permitted by concentrations will find weak spots and begin to cut. Under a velocity of about 30 feet per second, in earth material, the erosion would probably develop at a rather fast rate.

The model, as mentioned, however, does not consider the value of Bermuda grass sod or other types of lining to resist erosion. It considers the inlet conditions, and the irregular flow in the channel. Irregular flow is shown by the surface waves, piling up of the water on the outside of the curves, deflection of flow, and turbulence at the downstream end. All these irregularities indicate and would result in channel erosion. Studies of these conditions and irregularities indicate the manner in which the design might be improved, or where saving of cost of construction and maintenance might be obtained.

In order to show clearly the results, the flow over the spillway model crest was increased to a relative depth of about 12.7 feet. This is a greater depth than contemplated over the crest, but shows in extreme the conditions that would develop. The photographs ahow five arrangements of the spillway, and the cross-sections, shown on Drawings 4, 5, 6, 7 and 8, indicate measurements were taken for four conditions. The studies involved the following conditions:

I. Operation of the original spillway design and layout.

II. Operation of original spillway with levee on outside of downstream channel to confine the flow.

III. Operation in the spillway with the levee added under II, and with superelevation of the curve at the upstream end of the spillway, and with some widening of the inlet.

IV. Operation with the outer levee, no superelevation, and greater widening of the inlet than used for III.

V. Operation without levee or superelevation, but with improved inlet.

For Condition I, it is noted that surface waves exist, and that the depth of the flow increases on the outsides of the curves, and recedes on the insides. There would be a tendency to erode at the outsides of the curves during high flows, and along the toe of the dam.

For Condition II, it is noted that the depth of the flow at the upstream curve will be greater on the outside. The levee, however, has the effect of confining and improving the conditions in the downstream portion of the channel. Condition II approaches flow conditions desired, although further improvement appears necessary if it can be obtained. Surface waves are apparent.

For Condition III, it appears the superelevation has reduced the tendency of the depth to be increased on the outside of the upper curve because of the superelevation of the channel section, and that the flow approaches a more stable condition throughout the channel. Surface waves, a source of danger in unlined channels still are apparent. Conditions may also be improved by the improved inlet.

For Condition IV, where no superelevation is used, but the entrance was widened more than for III, thickening of the flow on the outside of the curves is noted, but the flow appears to be more uniform than for II, which was for a similar condition, but without widening of the inlet.

For Condition V, it is shown by the photographs that, although more flow is probably permitted for the same lake surface elevation, the channel flow will approximate that of Condition I.

## SURFACE WAVES

Surface waves are a usual phenomena in spillway channels. They usually occur where the flow is greater than the critical flow, and are caused by irregularities in the channels such as piers, piling, curves in the channel alignment, or other irregularities. For a channel layout similar to that for the Hicks project, it is believed the surface wave would exist for all substantial flows. The surface wave will cause erosion at points where it contacts the slope, if the slopes are not well protected by concrete, riprap, or other suitable permanent material. All the photographs show the surface wave in channel, and that the wave is reflected by the slope of the dam about one-half the way down the channel. The condition is principally a wave phenomenon, as the direction of the flow lines of the water is generally parallel to the channel alignment, except where centrifugal force causes a travel to the outside of the curves.

## OUTLET END OF CHANNEL

At the outlet end, a hydraulic jump, largely submerged, was formed. The tail water elevation was maintained by a weir downstream of the model. Although no damage was apparent to the model because of its construction, the erosion at that point would be considerable, because of the turbulence caused by the jump phenomenon. The point of occurrence of the hydraulic jump can be estimated by hydraulic formula where the shape of the channel remains stable. During progressive erosion the position changes, because of the constantly changing channel outlines.

PHOTOGRAPHS AND MOVING PICTURES

Photographs of the flow and model arrangement for each of the five conditions considered, as before mentioned, are included herewith. Many other views were prepared on slides for screen projection, were observed. A reel of moving pictures of the construction and operation of the model were also observed. It is understood that these data will be used for instruction purposes at the University of Texas.

The additional photographs and pictures show in more detail some of the features that have been described, and therefore, for an aid to the conclusions reached. The use of colored dye showed the channel velocity to continue in a reasonably straight line across the surface waves. The height of the last of the wave was several feet at the point of contact between the surface wave and the dam. The height should be about equal to the energy grade line, which would amount to the lake surface elevation, less the friction dissipated. The same condition was observed by the writer on actual discharge of the outlet works at Lake Kemp on the Wichita River soon after its completion in 1924. A similar condition was observed on a model of the Seminoe Dam, in Wyoming, in 1936. The occurrence of surface waves has been observed on several models, of which a model of the Alamogordo Dam in New Mexico, on the Pecos River, was a good example, because of piers at the crest, and the symmetrical arrangement.

It is believed the photographs and moving pictures were very helpful to indicate details that would have otherwise not been apparent, or certain. An example is the direction of flow across the surface wave, and another is extent of irregularity of flow. Close study of the moving picture helped to show the relative value of the changes made on the model to attempt to find improvement to structure layout.

## COMMENTS

The studies appear to indicate the following changes would result in a minimum of cost:

(a) Construction of a levee on the outside of the discharge channel.

(b) Superelevation of the channel sections.

(c) Widening of the spillway inlet.

The changes are stated in order of their estimated importance.

The comments on changes that are considered would be helpful are made only for channels that would be lined with concrete or other permanent lining. Under that condition it is believed regular flow might be approached with proper inlet and superelevation design. For earth lined channels or grass sod, it is not believed conditions would be greatly improved for the spillway being studies by making the changes proposed by the studies. Expanded studies with more topography and foundation information, and use of a straight uniform spillway channel, might indicate that a channel spillway would be successful without the use of permanent lining. The information available does not permit a recommendation of any value to be made, but there is apparently sandstone present at points in the foundation. Use of a straight channel at a suitable location might permit the most economical design if the possibilities were thoroughly explored.

## THE PURE OIL COMPANY PROJECT

The Pure Oil Company project is located on Spring Creek, a tributary of the Neches River in Van Zandt County. The location is about 3 miles southwest of Van. It is constructed of wetted and rolled embankment to a height above the flow line of the stream of about 20 feet. The surface area of the lake at spillway level is about 110 acres, and the capacity of the lake at that elevation is about 400 acre-feet. The drainage area is about 3 square miles.

The dam was constructed principally to impound water for municipal and industrial purposes. Although it will afford some measure of flood control, its value for that purpose is not great. The need for flood control in the immediate vicinity downstream of the dam is likewise not of much importance.

The model study may indicate the advantages afforded by a concrete overflow or ogee spillway, provided such a spillway can be as economically constructed. The model may also indicate economic answers to the following:

(a) Shape of walls at the ends of the spillway.

(b) Length and best position of the stilling pool.

(c) Depth of maximum overflow to produce optimum results.

(d) Improvements necessary downstream of the spillway.

The above questions will affect almost the entire economic design of the dam. It is quite possible, the most economic design has been obtained, which required experience and research the extent of which may have been considerable, and even a model may have been constructed and operated.

MODEL

The model has been constructed and set in place ready for operation. Its operation under an unknown amount of flow was observed.

COMMENTS

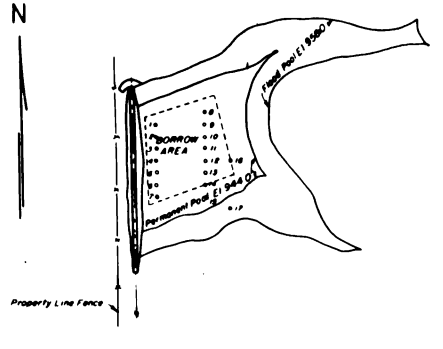
The conditions at the spillway crest appeared excellent. Due to the sloped end walls, a concentration of flow occurred at the ends of the mass spillway section. The concentration may cause erosion of the concrete at that point, and negative pressures could possibly exist. There was disturbance of a usual nature in the stilling pool. More study might show that some revision of the end or head wall design would have been helpful, and that the width or depth of the stilling pool may have been changed for economy. These comments are made, however, without the aid of extensive study of the model operation, or of the design as it now exists. The dam has been built and has operated successfully for several years.

The main purpose of the model was to obtain information on spillway structures of this type that will be of help in preparing economical designs of other structures in the State of Texas.

A H Woolverton

A. H. Woolverton
Consulting Engineer

AHW:rk




## GENERAL PLAN OF RESERVOIR

| Elevation | Surface Area | Storage * Ac Ft | Storage * Inches |
|---|---|---|---|
| 944 | 21 | 136 | 1.72 |
| 947 | 29 | 208 | 2.63 |
| 951 | 41 | 348 | 4.40 |
| 955 | 58 | 546 | 6.92 |
| 956 | 63 | 607 | 7.72 |

Top of Dam El 961.0

Spillway Crest El 956.0

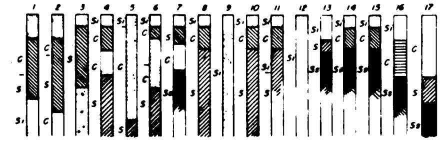
12" Outlet Structure El 944.0

Drainage Area 950 Ac

Time of Concentration 73 Min

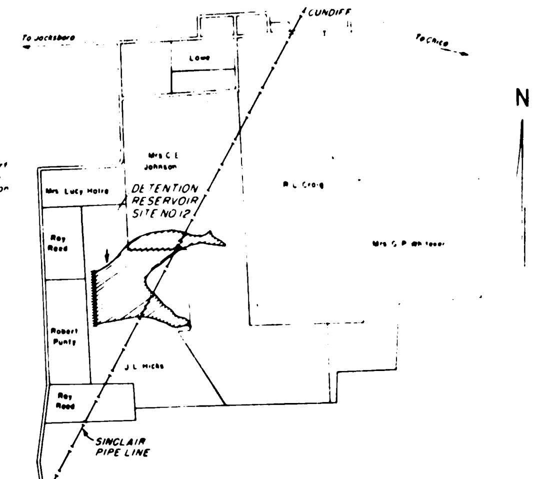
* Includes Volume of Borrow

Sta 0+00 on the ℄ of the Dam of Site No 12, West Fork of the Trinity is located on a line bearing N 26°-00 W a distance of 1663 feet from the South West corner of the Charles Hartman Survey. The SW corner of the Charles Hartman Survey is fully described in Abstract No 1390 of the Deed Records on file in Jack County, Texas



## LOG OF BORINGS IN BORROW AREA

LOCATION ON GENERAL PLAN




## VICINITY MAP

0 ¼ ½ 1

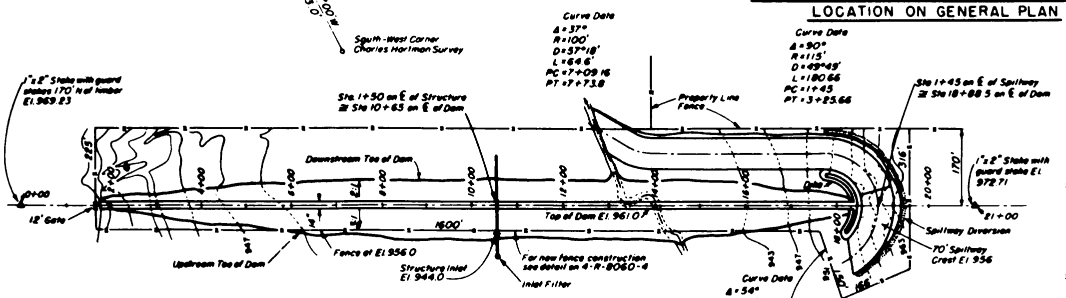
SCALE OF MILES

I certify that this reservoir plan correctly represents the results of surveys made by me on the 27 day of November 1950 and that the application by J.L. Hicks which it accompanies is dated April 18 1951

Surveyor

Agricultural Engineer

U S D A Soil Conservation Service




Minimum of 6 inches of topsoil to be placed on downstream and upstream slopes of dam, top of dam, bottom and side slopes of spillway, and side slopes and top of dikes in accordance with specifications.

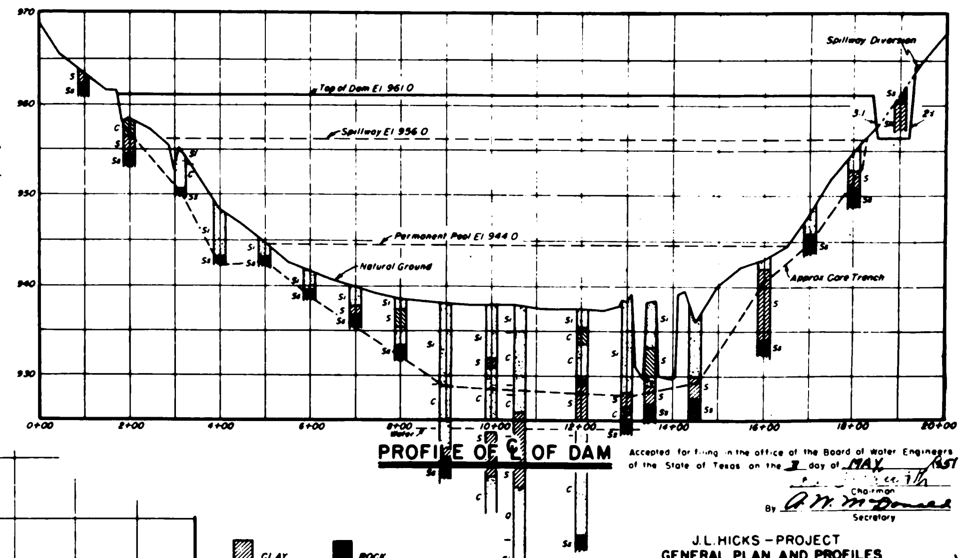
Stream channel within embankment area to be cleared of objectionable material in accordance with "Special Stream Channel Excavation" of the specifications.

Spillway Diversion: 18" Effective height, 2:1 side slopes, minimum base 10 ft. Cost of diversion to be included in price bid per cu yd for "Unclassified Channel Excavation."

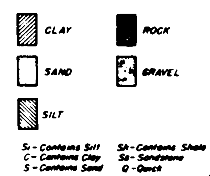
## PLAN OF EMBANKMENT & SPILLWAY

100 0 100 200 300 400 500 600

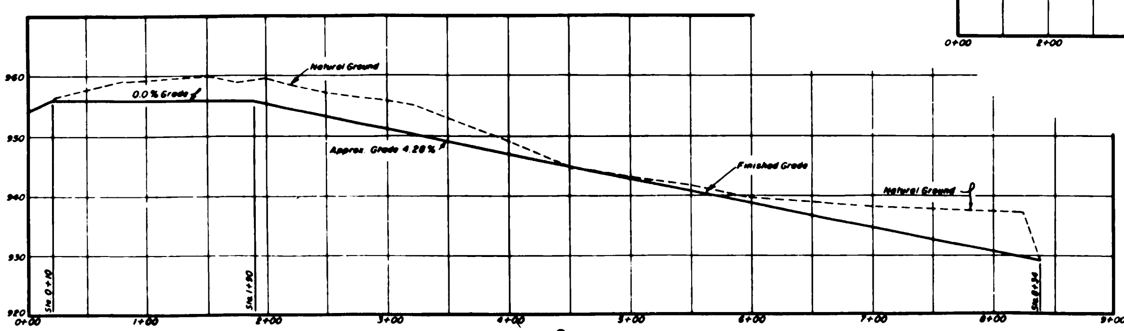
SCALE IN FEET




## PROFILE OF ℄ OF DAM




Si - Contains Silt
C - Contains Clay
S - Contains Sand
Sh - Contains Shale
Ss - Sandstone
Q - Quick




## PROFILE OF ℄ OF SPILLWAY

Accepted for filing in the office of the Board of Water Engineers of the State of Texas on the 3 day of MAY 1951

Chairman

By A. W. McDonald

Secretary

Dwg. 1

FOR-IN SERVICE USE ONLY

J.L. HICKS – PROJECT

GENERAL PLAN AND PROFILES

DETENTION STRUCTURE SITE NO.12

WEST FORK OF TRINITY RIVER

ABOVE LAKE BRIDGEPORT

SUB-WATERSHED NO. 1 OF THE TRINITY RIVER

TEXAS

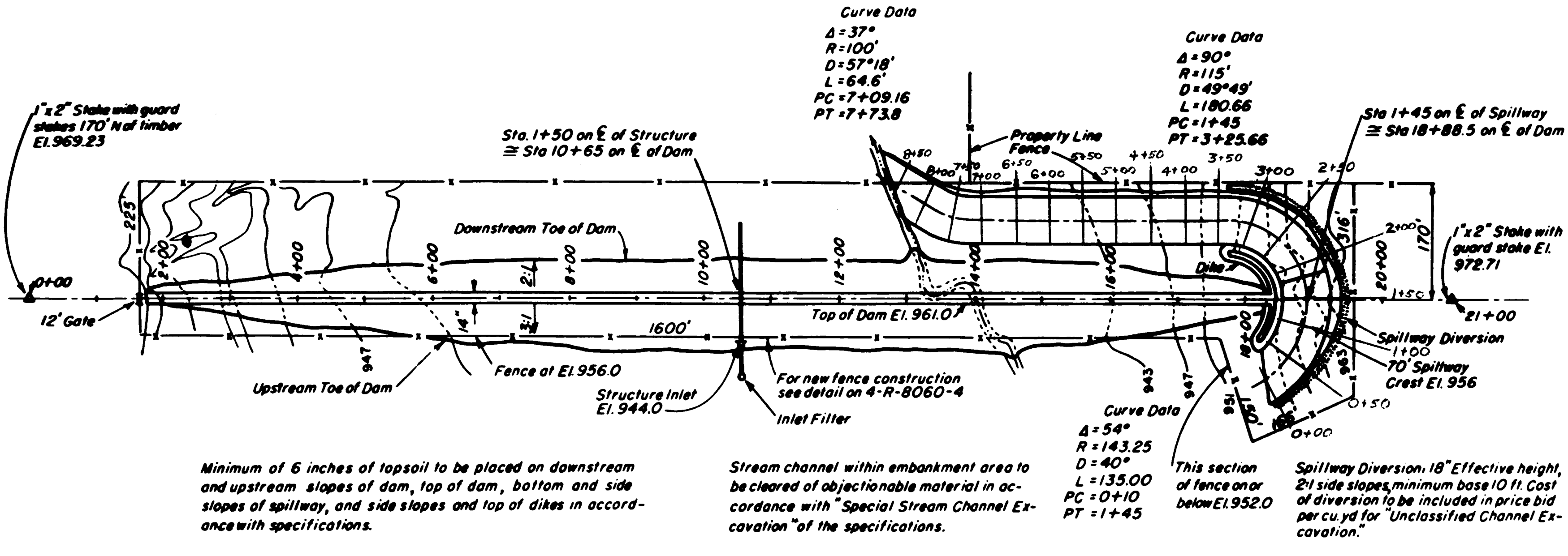
U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

H H BENNETT - CHIEF

REGION 4 DIRECTOR - LOUIS P. MERRILL

| REFERENCE | | | | |
|---|---|---|---|---|
| CARTOGRAPHIC APPROVAL | | | TECHNICAL APPROVAL | |
| COMPILED | TRACED | CHECKED | DATE | |
| J.N.Y. | R.D.B. | J.N.Y. | 3-22-51 | 4-R-8060-1 |

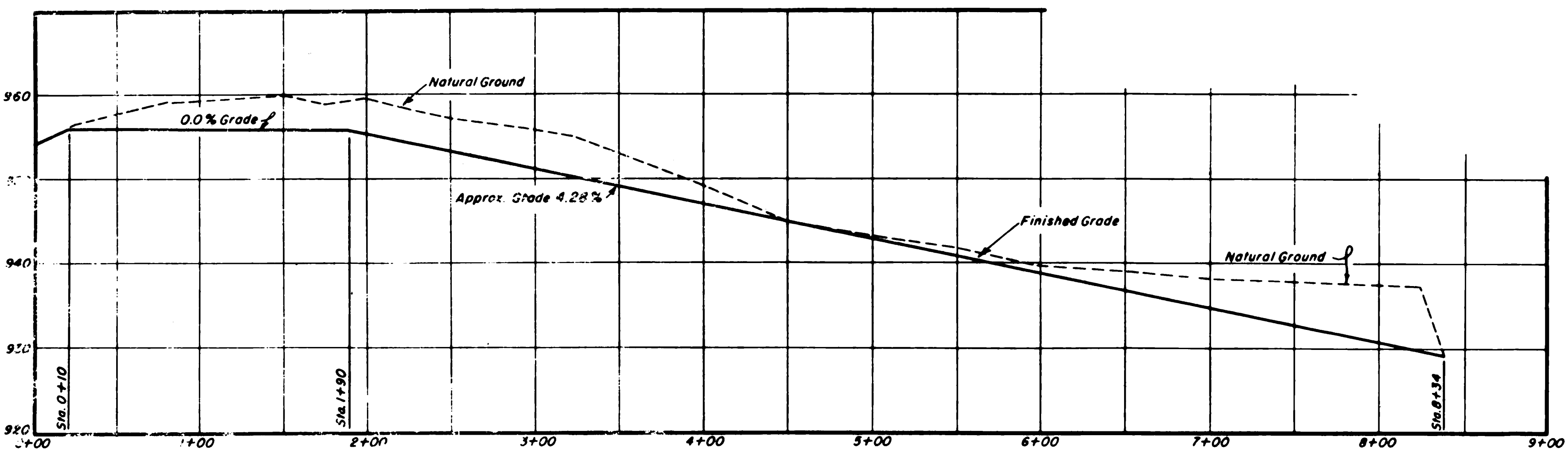



Minimum of 6 inches of topsoil to be placed on downstream and upstream slopes of dam, top of dam, bottom and side slopes of spillway, and side slopes and top of dikes in accordance with specifications.

Stream channel within embankment area to be cleared of objectionable material in accordance with "Special Stream Channel Excavation" of the specifications.

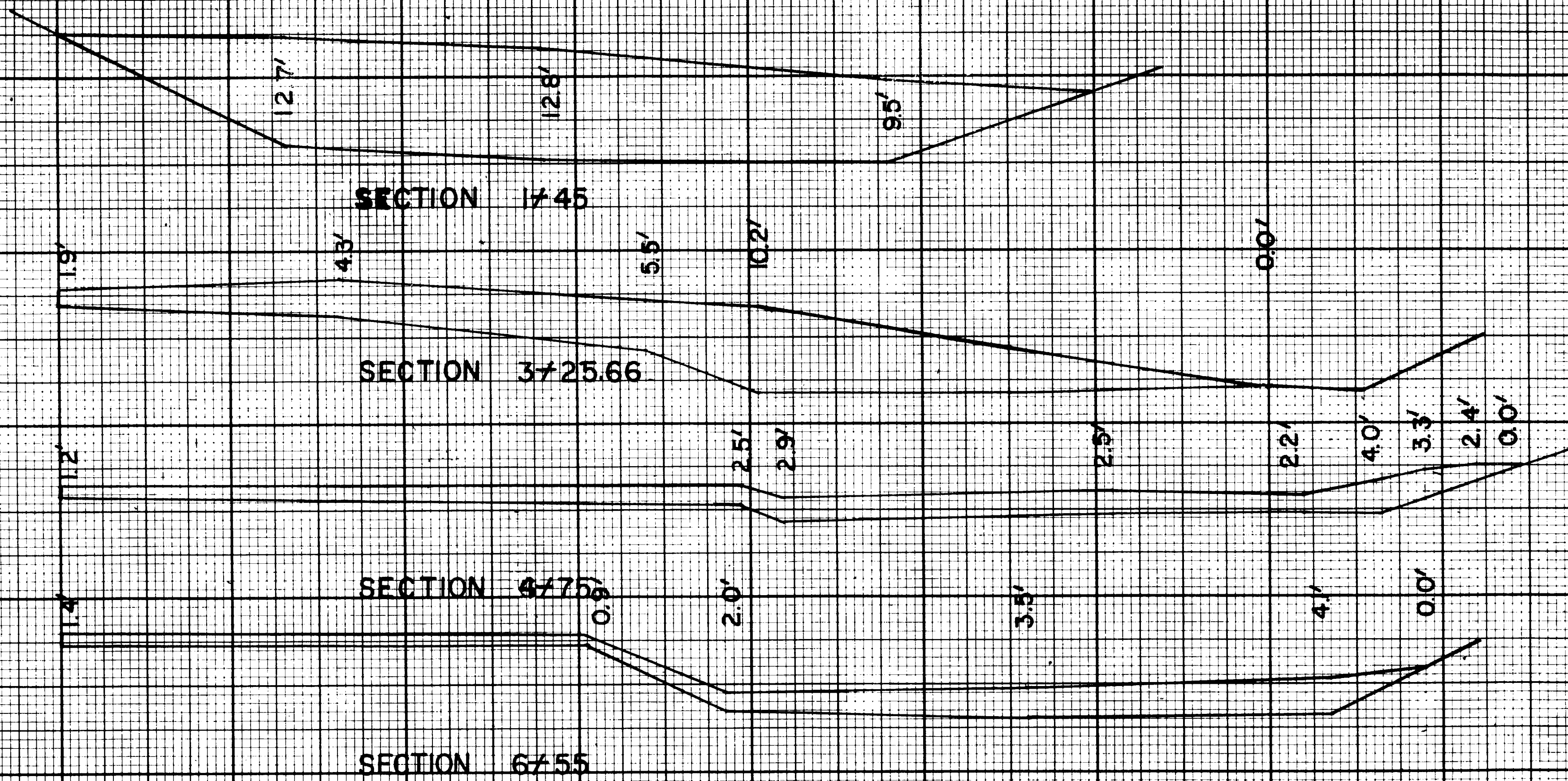
## PLAN OF EMBANKMENT & SPILLWAY

Dwg. 2.




**PROFILE OF ℄ OF SPILLWAY**

Dwg. 9.



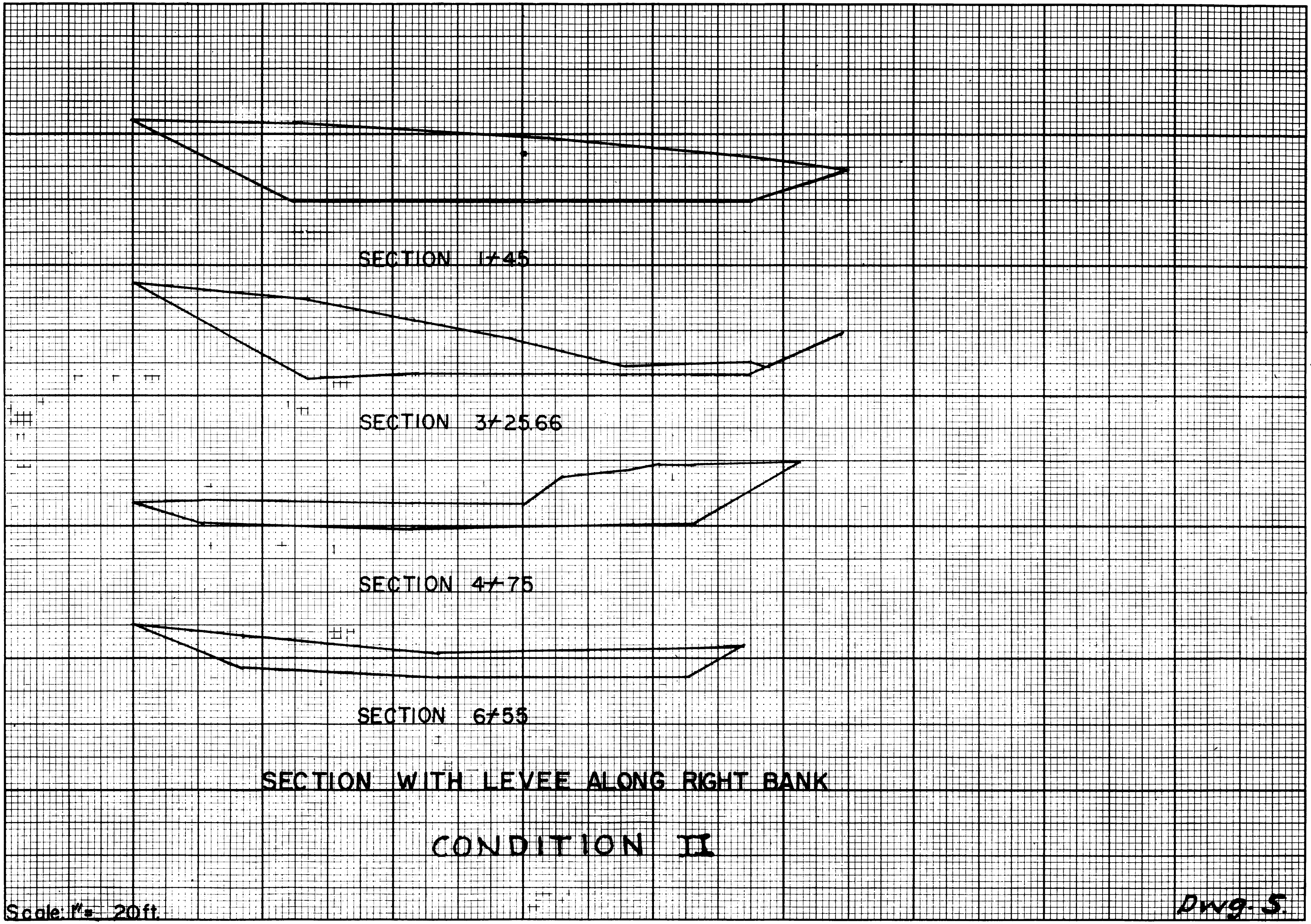

CROSS-SECTIONS OF SPILLWAY CHANNEL FLOW ON MODEL OF STRUCTURE

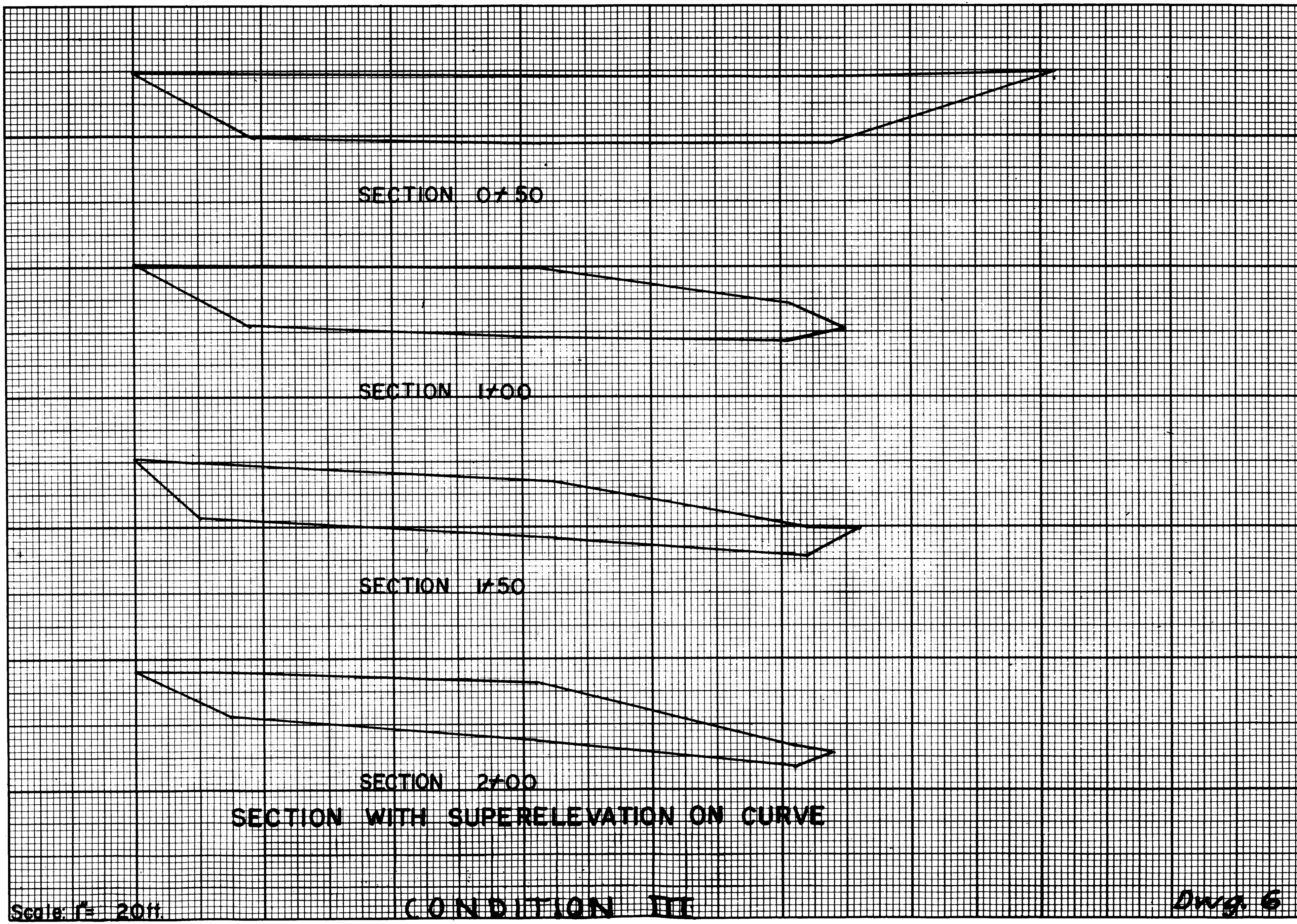
AS BUILT

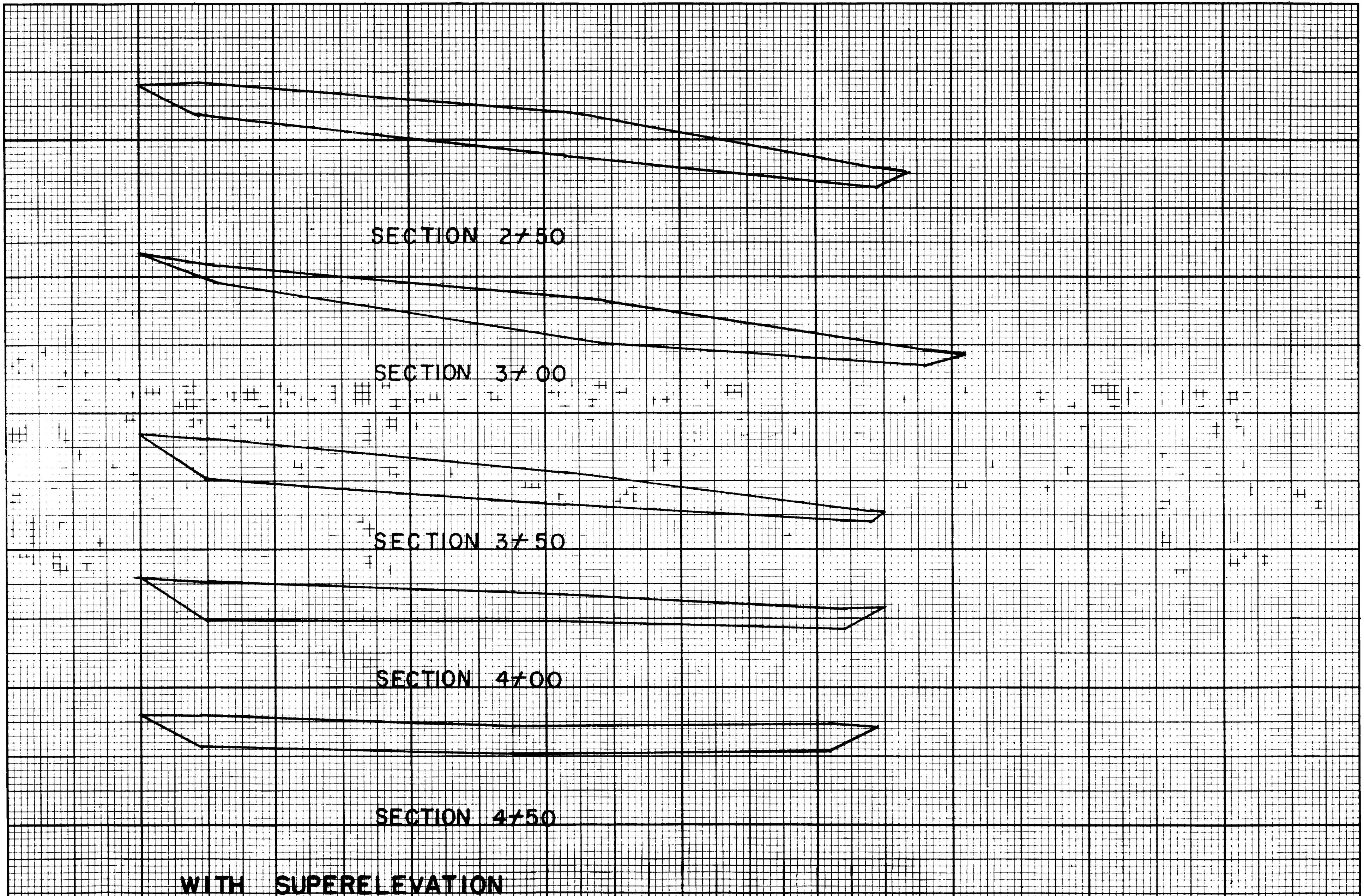
CONDITION I

Scale: 1″ = 20 ft.

Dwg. 4


SECTION 1+45
SECTION 3+25.66
SECTION 4+75
SECTION 6+55
SECTION WITH LEVEE ALONG RIGHT BANK
CONDITION II
Scale: 1" = 20 ft.
Dwg. 5.

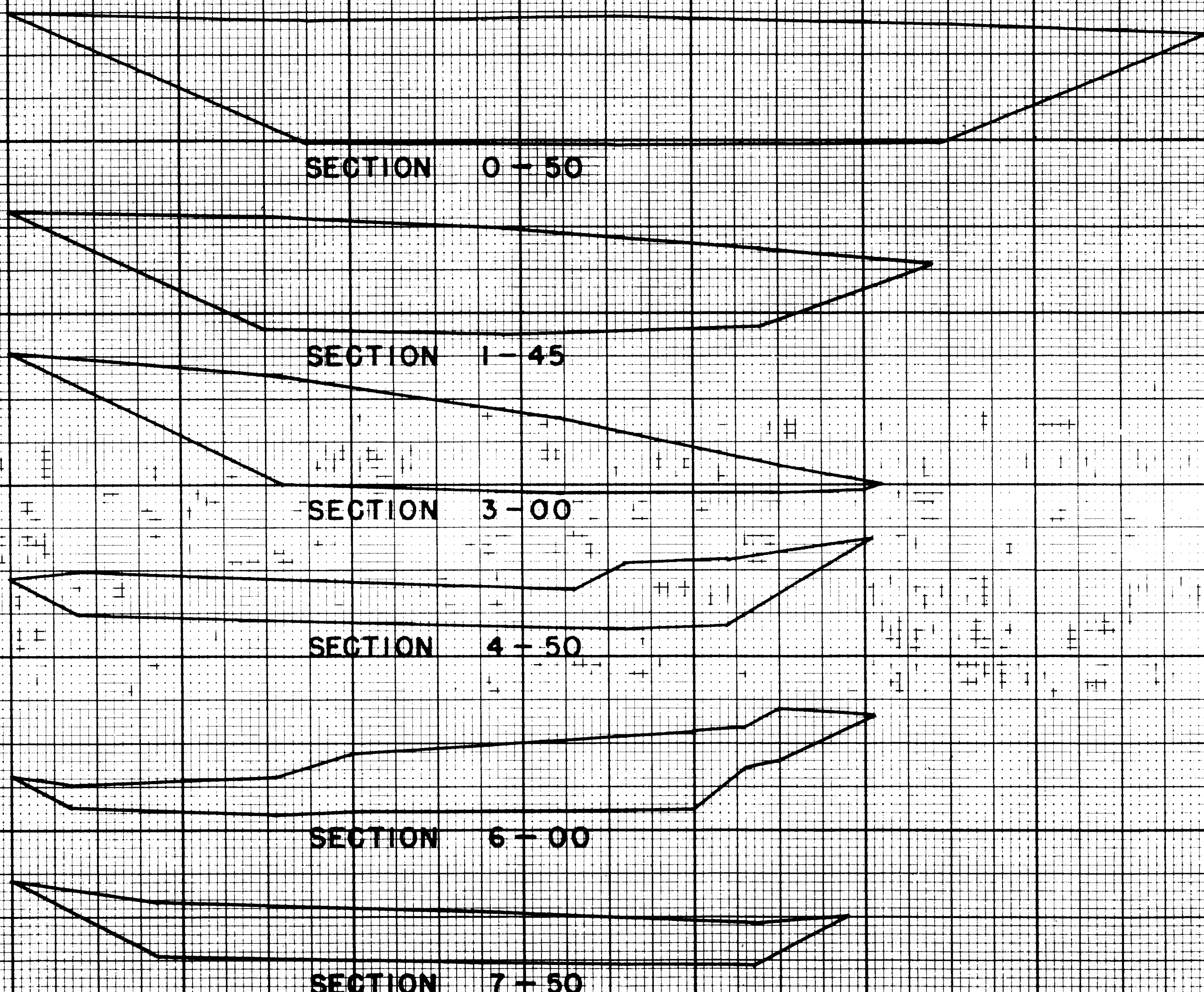

SECTION 0+50
SECTION 1+00
SECTION 1+50
SECTION 2+00
SECTION WITH SUPERELEVATION ON CURVE
Scale: 1" = 20 ft.
CONDITION III
Dwg. 6




Scale: 1" = 20 ft.

CONDITION III

Dwg. 7.

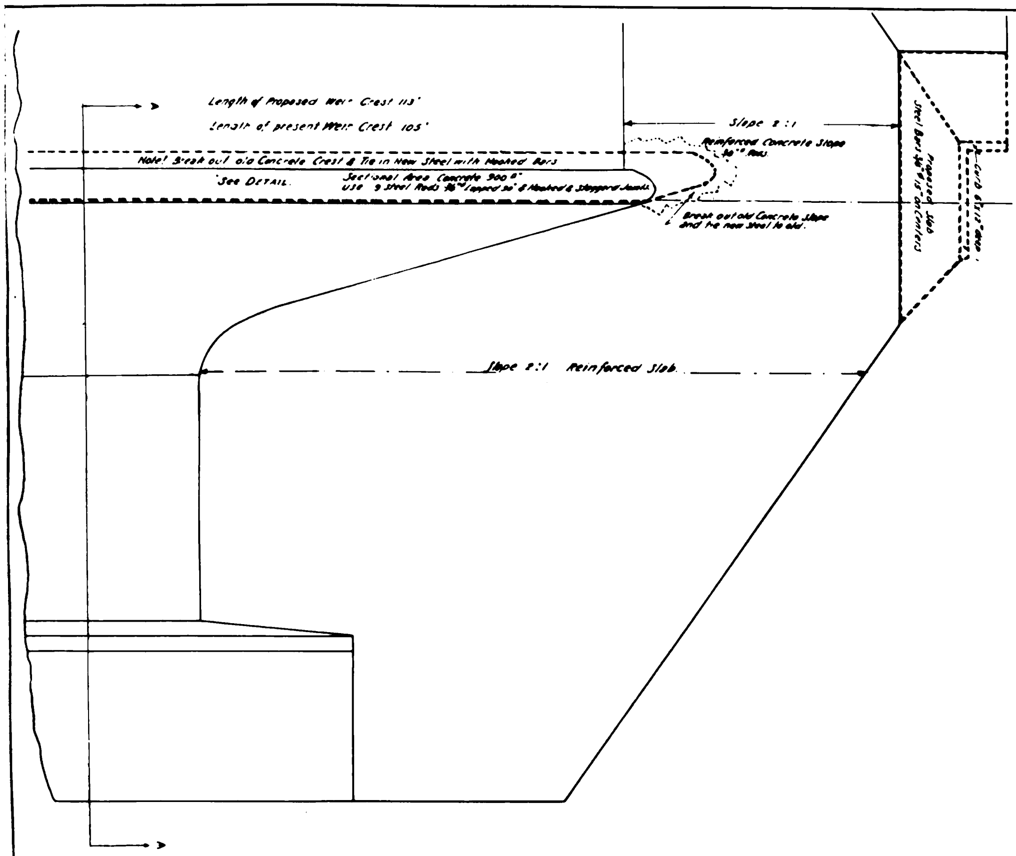



SECTION WITHOUT SUPERELEVATION AND WITH ALTERED ENTRANCE

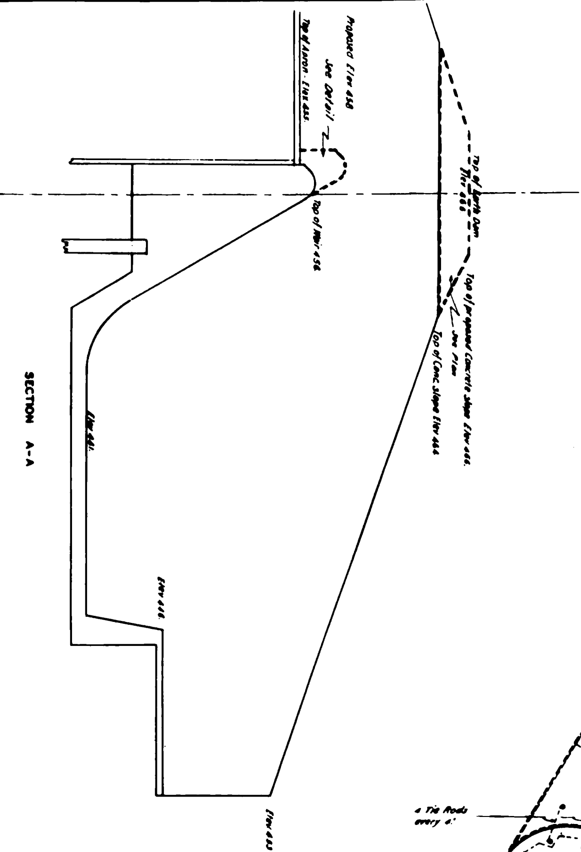
CONDITION IV

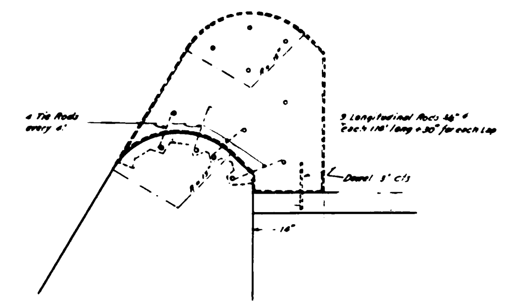
Scale: 1" = 20 ft

Dwg. 8.

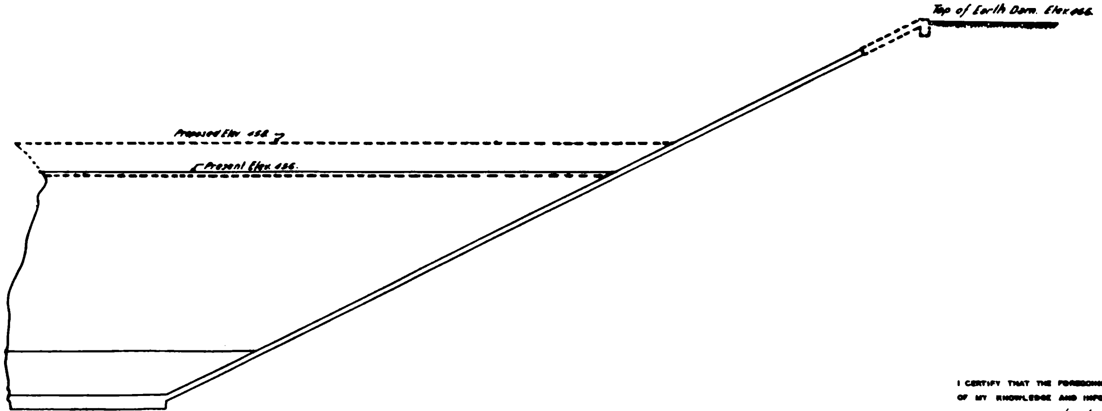



HALF PLAN 1" = 4.0'




DETAIL SECTION OF PROPOSED WEIR CREST.
SCALE 1"= 1'




HALF LONGITUDINAL SECTION SCALE: 1" = 4.0'

I CERTIFY THAT THE FOREGOING IS TRUE AND CORRECT TO THE BEST OF MY KNOWLEDGE AND INFORMATION. DATED ________ 19__

________
POST OFFICE ADDRESS

# THE PURE OIL COMPANY

## PROPOSED CHANGES IN PRESENT SERVICE SPILLWAY OF RHINES RESERVOIR, PERMIT Nº. 1462. VAN ZANDT COUNTY, TEXAS.

TO INCREASE STORAGE CAPACITY OF RESERVOIR

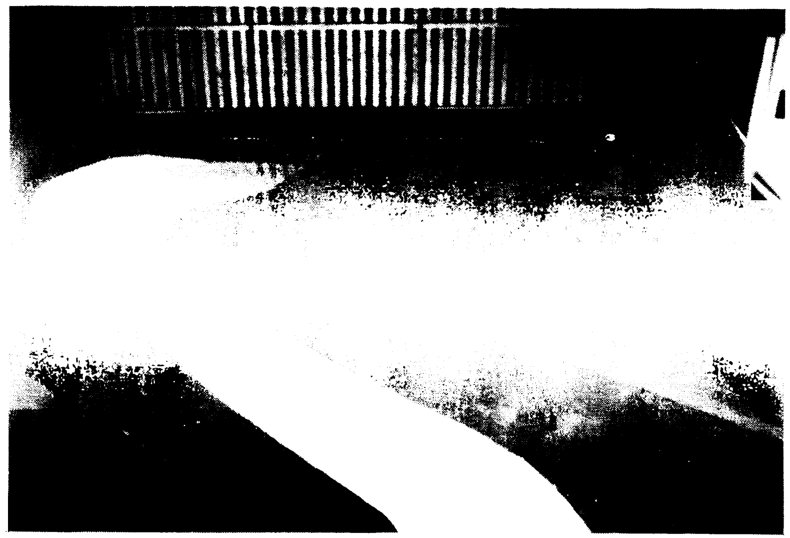
APPLICATION BY THE PURE OIL COMPANY

ACCEPTED FOR FILING IN THE OFFICE OF THE BOARD OF WATER ENGINEERS OF TEXAS ON THE ____ DAY OF ________ 19__

________ CHAIRMAN

________ SECRETARY

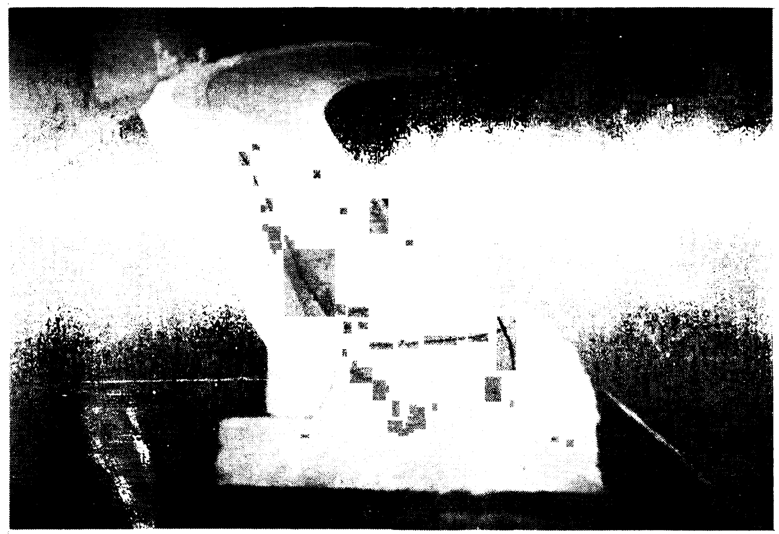
BWE 1779-3 SHEET NO. 3 OF 3 SHEETS Dwg. 9



I

Model as originally built

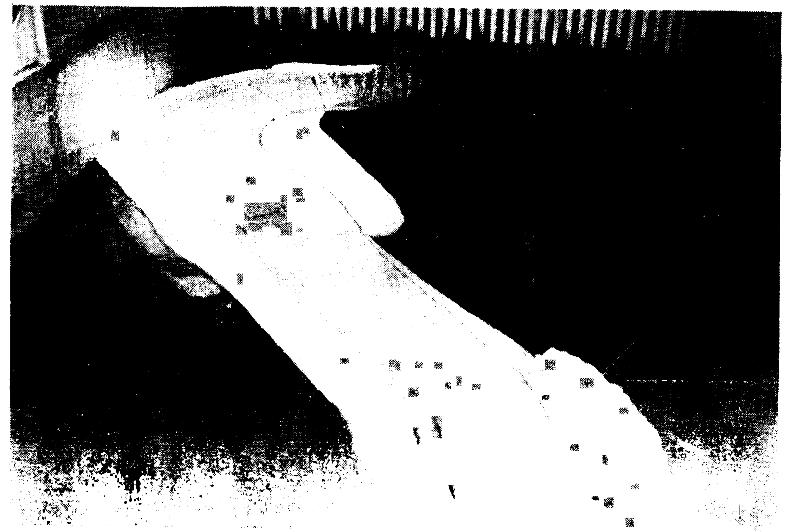
II-a.



II-b.

Model with outer levee to contain spill

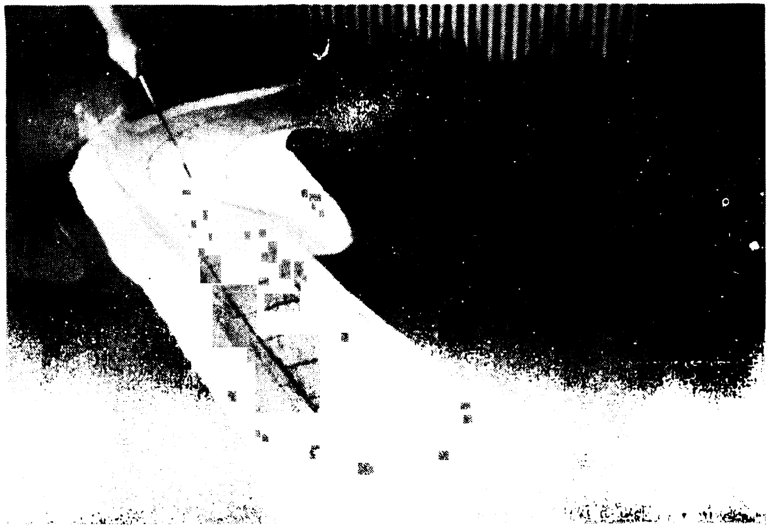
III-a.



III-b.

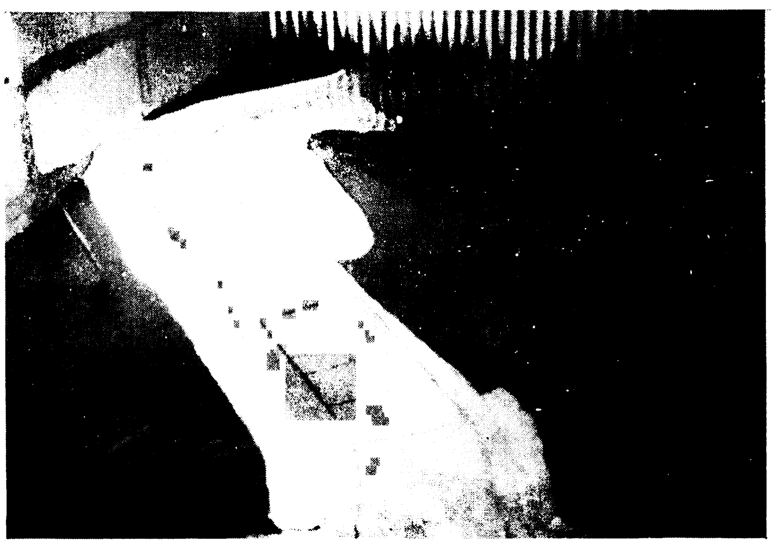
III-c.

Model with outer levee, superelevation and slightly improved entrance



IV-a. Die pattern shows bottom currents

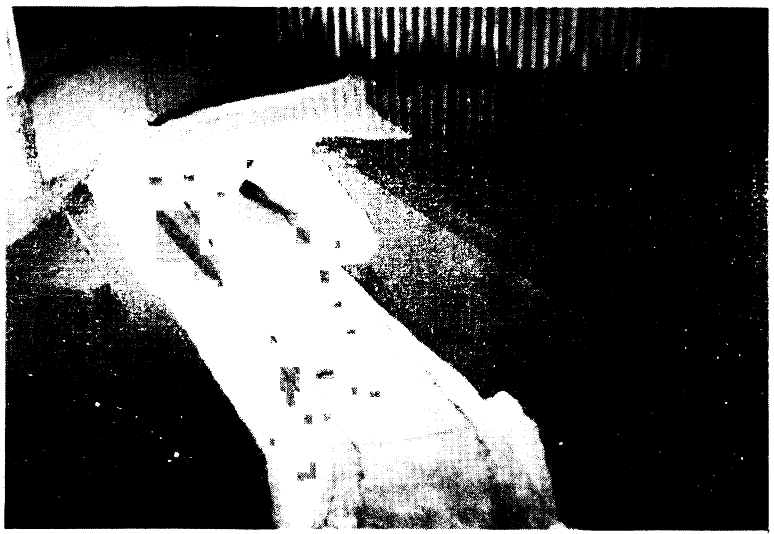
IV-b. Die pattern shows bottom currents



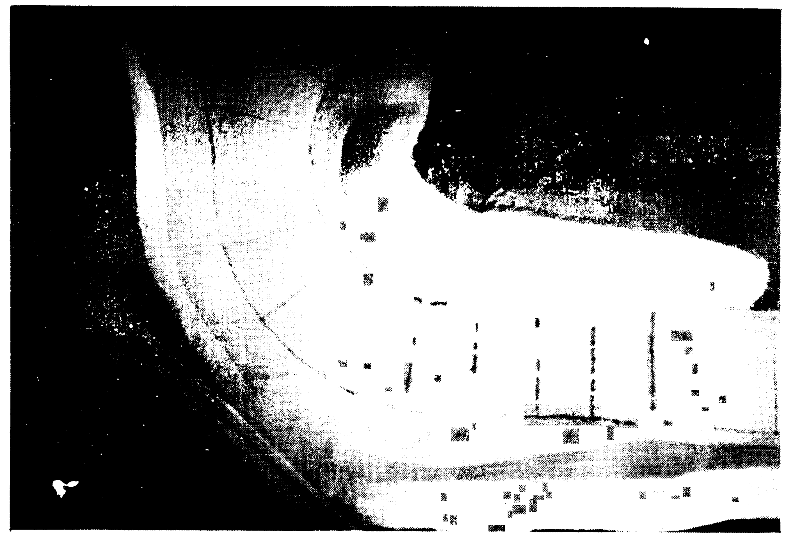
IV-c. Drops of die show surface current

IV-d. Drops of die show surface current

Model with outer levee, no superelevation but with greatly improved entrance

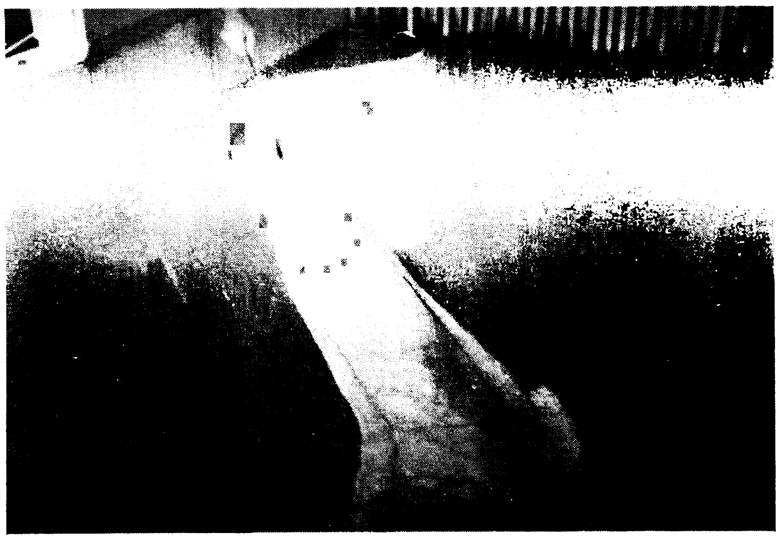


IV-e. Dark spot die shows area where spillway is nearly dry.

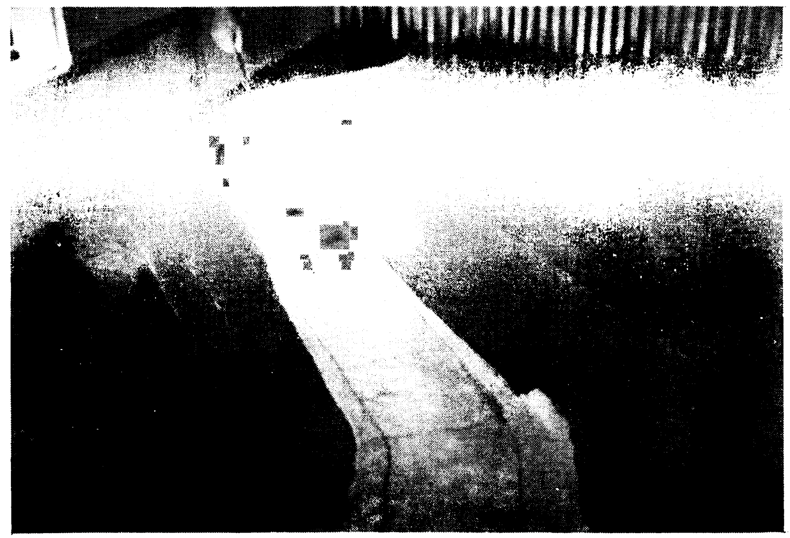


IV-f. Closeup of improved entrance.

Model with outer levee, no superelevation but with greatly improved entrance



V-a. Die pattern shows bottom current



V-b. Drops of die show surface current

Model without levee or superelevation but with improved entrance

MODEL STUDY FOR THE LUCY MAE KUHN DAM PROJECT

GENERAL

Tests were performed on a model of a reinforced concrete dam to be constructed for Mrs. Lucy Mae Kuhn, Rowena, Texas. The dam is to be located on the Colorado River, approximately six miles west of Ballinger, Texas. (See Fig. 1)

PURPOSE

The purpose of the tests was to study the hydraulic characteristics of the Kuhn dam and appurtenances under flow conditions commensurate with a head of 5.5 feet above the top of the dam. The determination of the assumed head is the result of calculations by W. O. Leach of Coleman, Texas, designing engineer of the proposed dam.

In particular, tests were conducted to determine the erosion and scouring effects on natural ground or compacted fill due to the action of stream flow at various velocities. Available facilities prohibited the testing of the structural soundness of the dam.

MODEL

The model of the Kuhn dam was constructed to a scale ratio of 24 to 1. Since no tests were conducted concerning the structural properties of the dam, the main section of the dam and the right and left wing sections were constructed of wood. The fill, simulating the shape of the river banks and the flood plain, was composed of various ratios of sand to cement. (See Figs. 2, 3, and 4)

## PROCEDURES

Calculations to determine a tail-water rating curve for the model were made by Dr. Walter L. Moore, Department of Civil Engineering at the University of Texas. For a head of 5.5 feet (0.229 feet on the model) corresponding to a flow of 3,440 cfs (1.47 cfs on the model), a tail-water depth of 0.473 feet was used on all tests. Tests were generally run in increments of 30 minutes duration.

The initial fill was composed of a mixture of sand and cement in the ratio 50:1. After operating the model for one and one half hours with this ratio there was no observable evidence of erosion. With this stable mixture in place, cross-sectional measurements were made. In addition, measurements were made using a tail-water depth of 0.41 feet. At this depth a hydraulic jump of considerable turbulence was formed. The measurements are shown in Figs. 5, 6, and 7.

Subsequent tests were made using sand-cement ratios of 110:1 and 150:1 with no appreciable evidence of erosion. Tests of sand-cement ratios of 300:1 and 500:1 indicated the practicable limits of the rate of erosion for the Kuhn model. The ratio of 300:1 showed visible evidence of erosion after a 30 minute test (See Fig. 8) but was too resistant to erosion at low velocity flow. The fill composed of the 500:1 ratio washed out completely within 6 minutes, thereby greatly exceeding the limits for practicable study. (See Fig. 9)

Since a mean-value of the two previous ratios seemed likely to be the desirable mixture, the model was rebuilt to test a ratio of 400:1. Figs. 10 and 11 show the results of this test which were indicative of the critical points of erosion with respect to this particular design.

In each test, a layer of the sand-cement ratio to be tested was placed along the toe of the dam. Only slight evidence of scour was observed using the ratio of 75:1, partial scour for the ratio of 110:1, and almost a complete washout with the ratio 150:1. This substantiates the known fact that the most severe scouring action occurs at the toe of an overflow dam.

## CONCLUSIONS

The tests disclosed several points of weakness which should be considered in the design of the Kuhn dam. As evidenced by the model tests, extensive erosion would occur along the downstream face of each wing section if protective riprap or a concrete apron were not provided. It is the opinion of the testing engineers that the purpose of the dam could best be served if the wing walls are raised to a height of six to eight feet above the top of the dam. This would allow passage of the 5.5 feet of head assumed by Mr. Leach. Also, a vertical section perpendicular to the wing wall should be constructed extending downstream a sufficient distance to provide protection against the turbulence resulting from the hydraulic jump. This will form, in effect, a rectangular weir capable of discharging virtually all expected high flows. Since the river banks on the downstream side of the wing walls should be protected with riprap, it would be advisable to provide further overflow capacity to the dam by extending the wing walls upward into the river banks to the height of the surrounding ground land.

It is believed that the tests show only the relative degree of severity of erosion in its early stages. No attempt was made to determine the rate at which erosion would advance.

John P. Dougherty

John P. Dougherty
Hydraulic Engineer

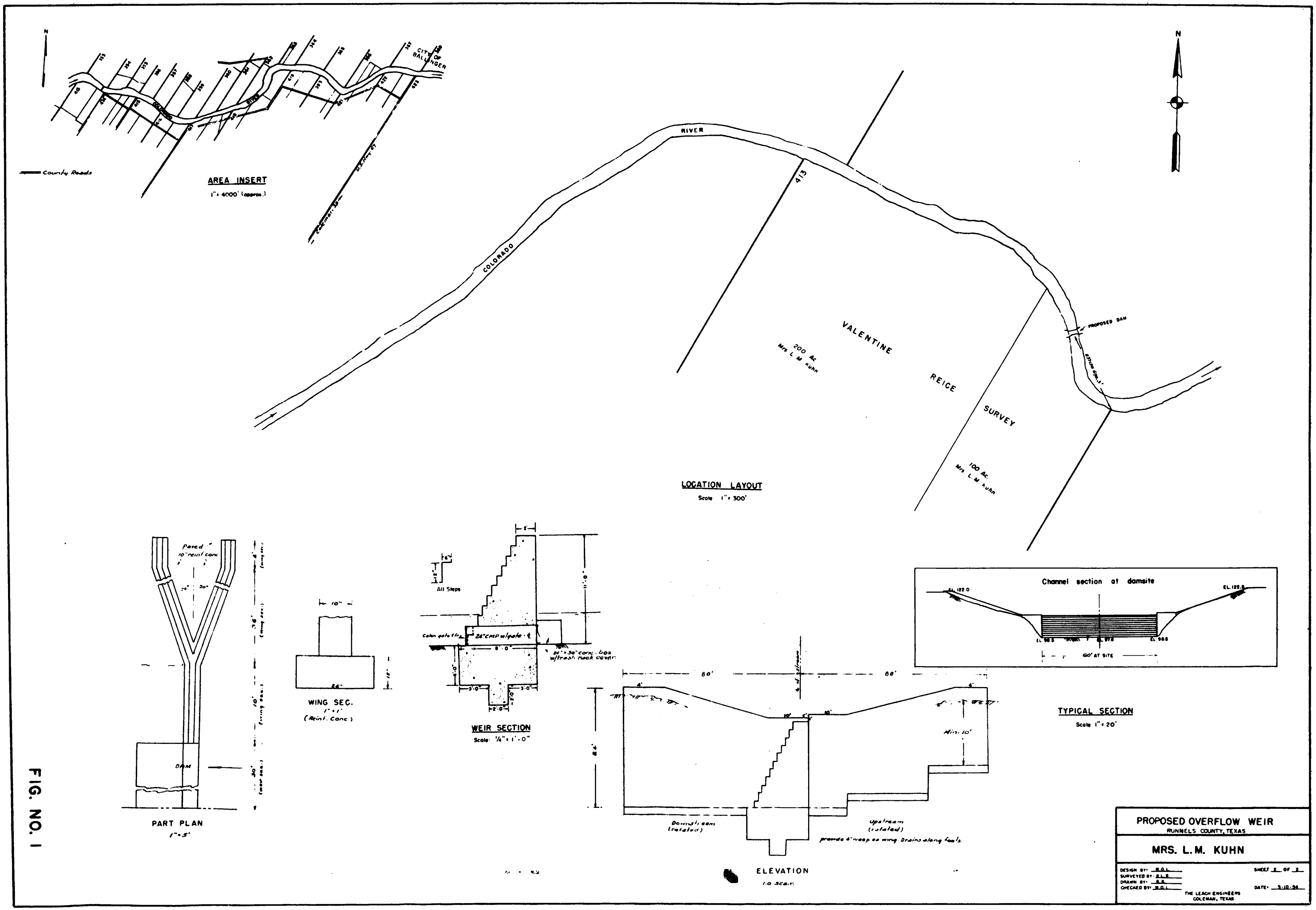



FIG. NO. 1

B.W.E.- 1847

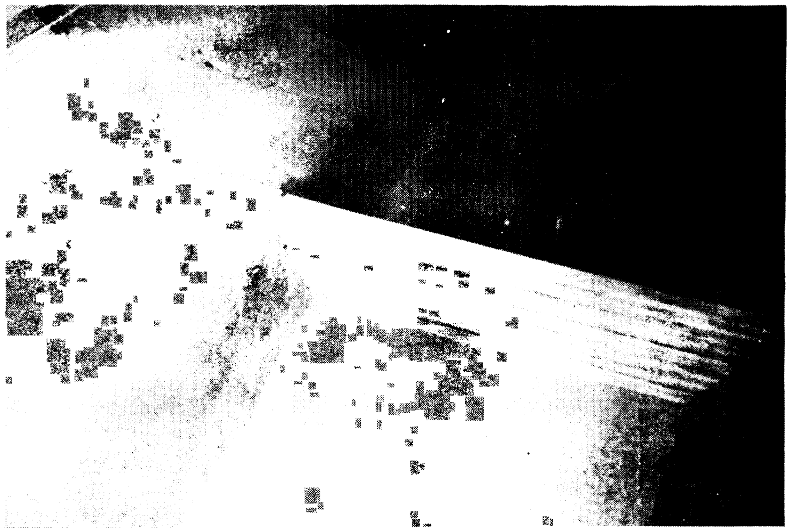


Fig. 2

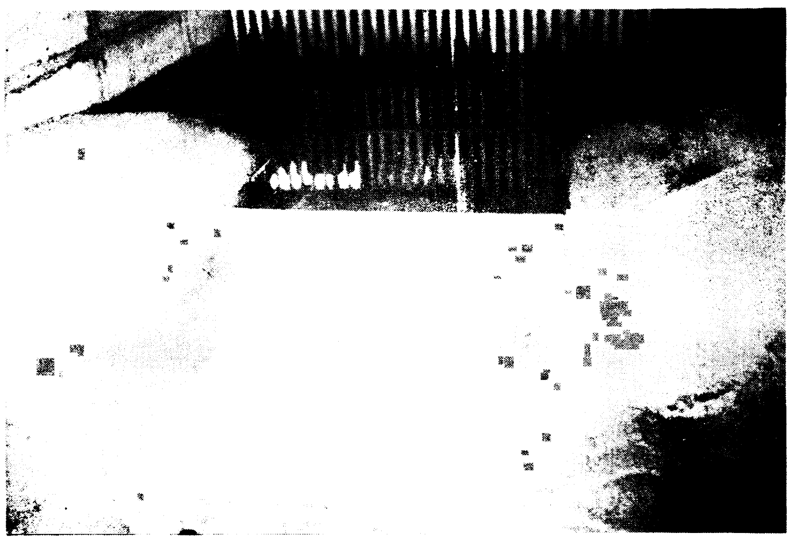


Fig. 3

Model as originally built.

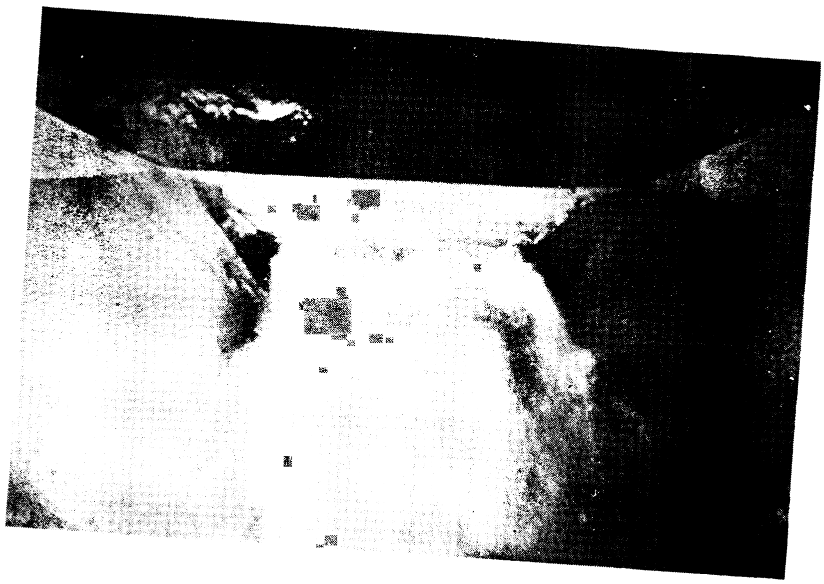
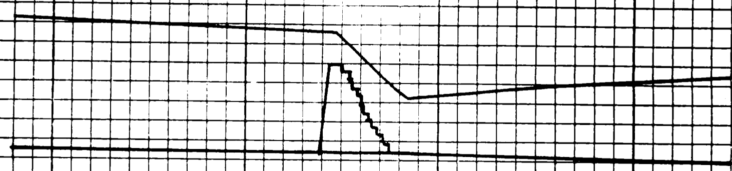
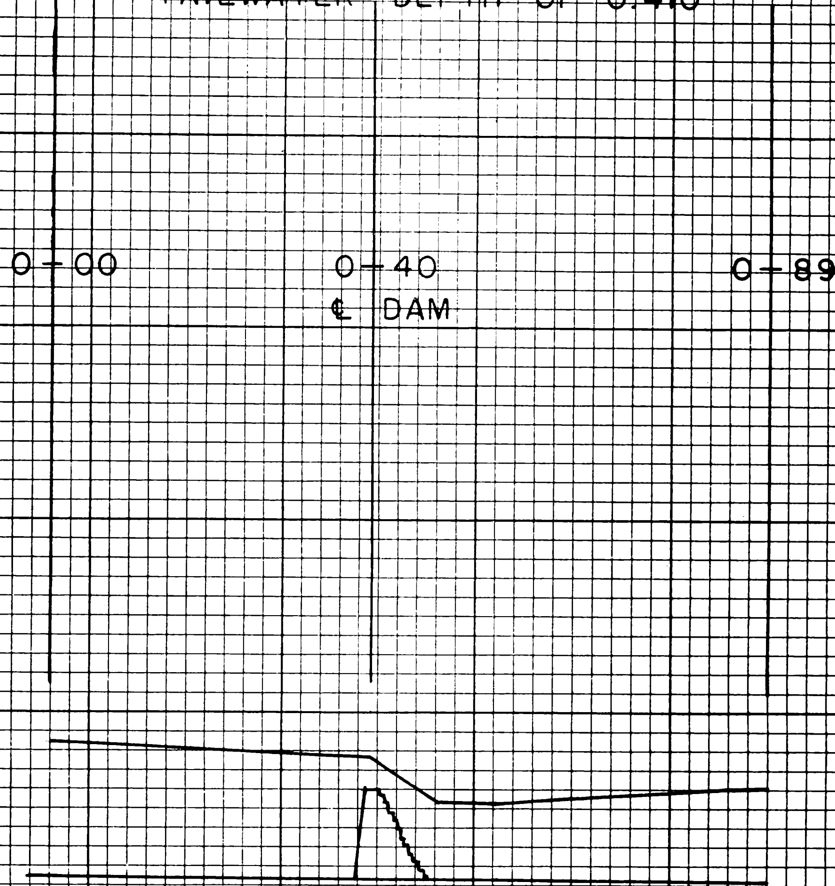


Fig. 4

Model in operation during tests.



LONGITUDINAL SECTION AT ℄ OF DAM
TAILWATER DEPTH OF 0.410'




LONGITUDINAL SECTION AT ℄ OF DAM
TAILWATER DEPTH OF 0.473'

Scale: 1" = 24'

FIG. NO. 5

9-1-54

## CONDITION OF FLOW WITH TAILWATER DEPTH AT 0.410'

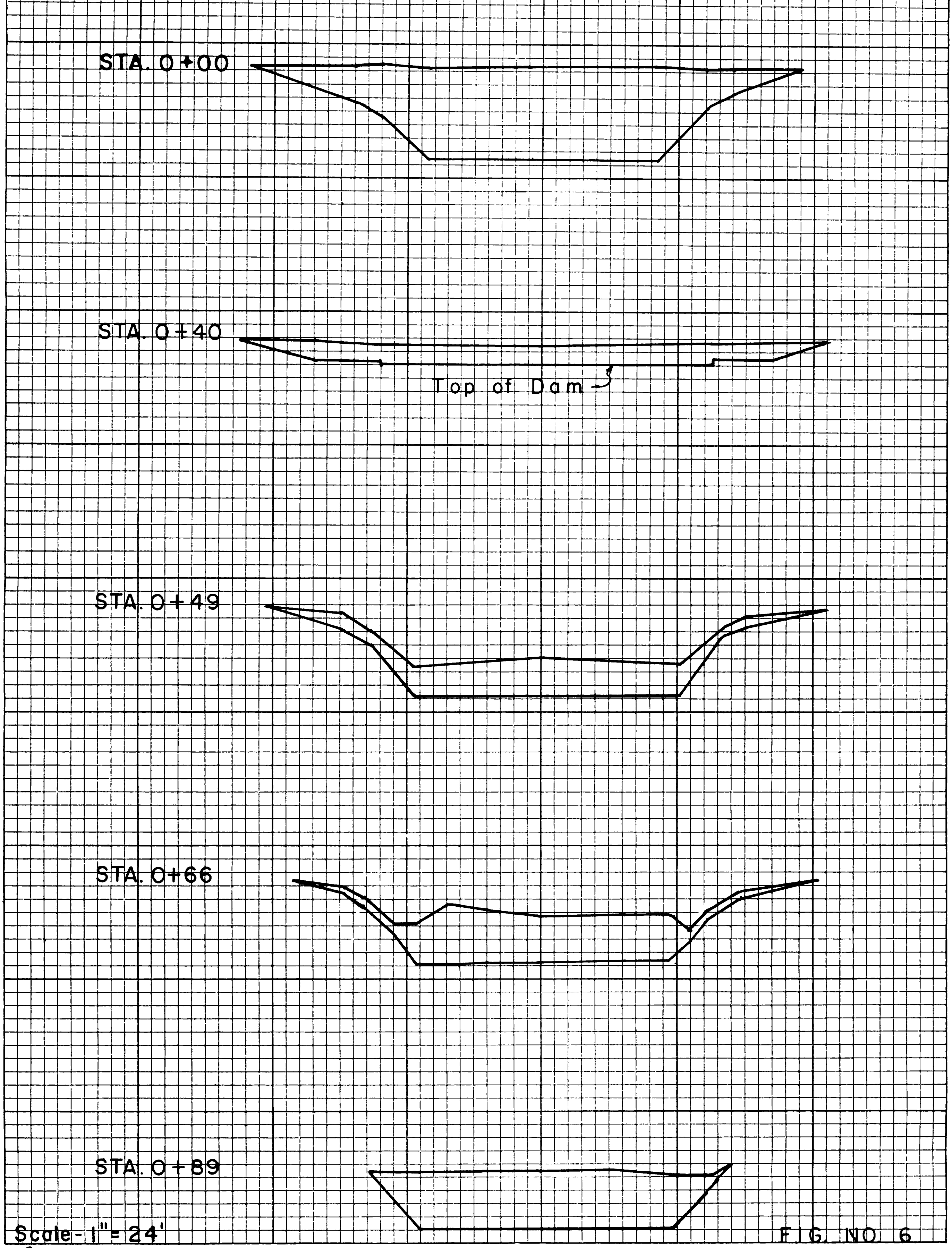
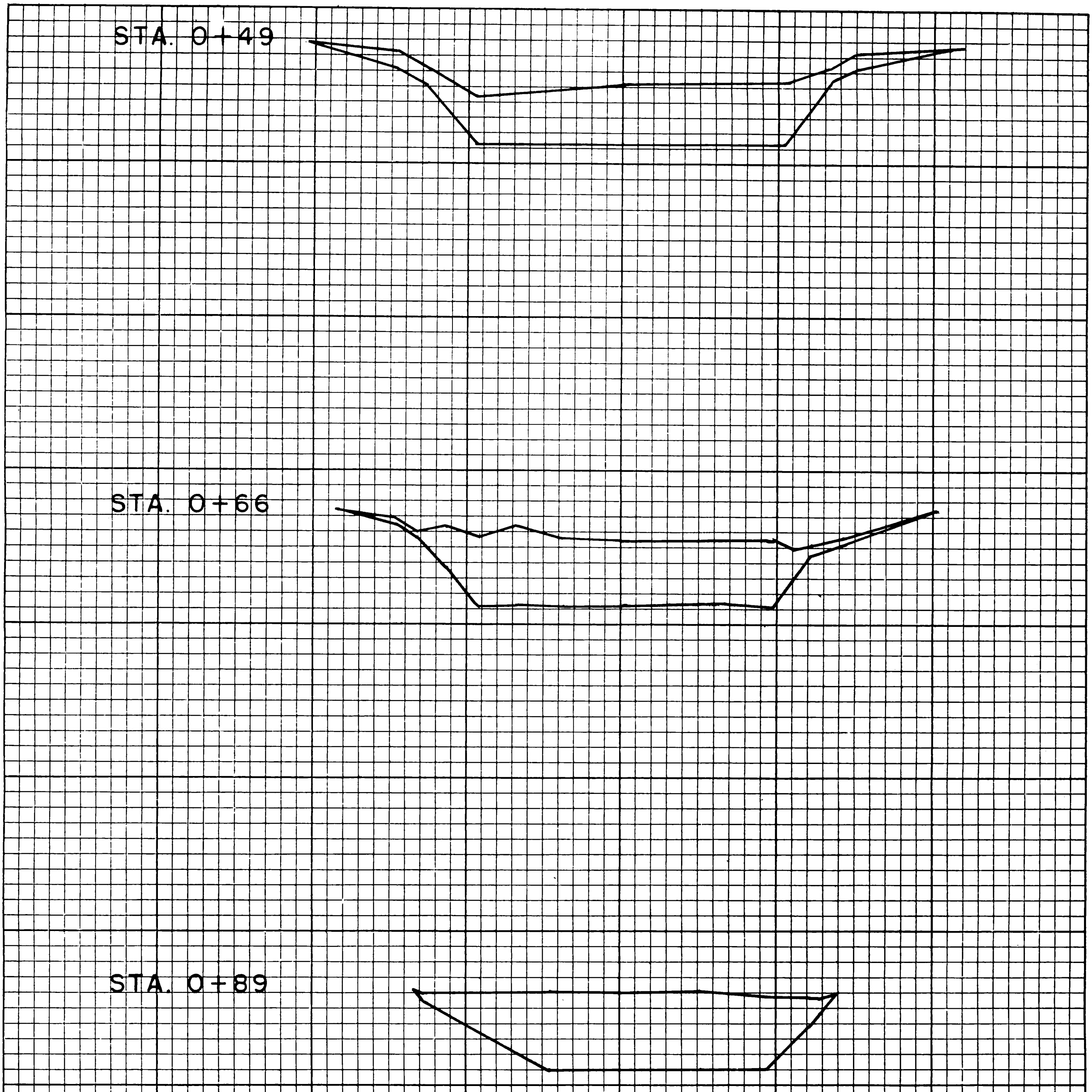



FIG. NO. 6

GLL 9-1-54




CONDITION OF FLOW WITH TAILWATER
DEPTH AT 0.473'

Scale: 1" = 24'

FIG. NO. 7

GHd 9-1-54

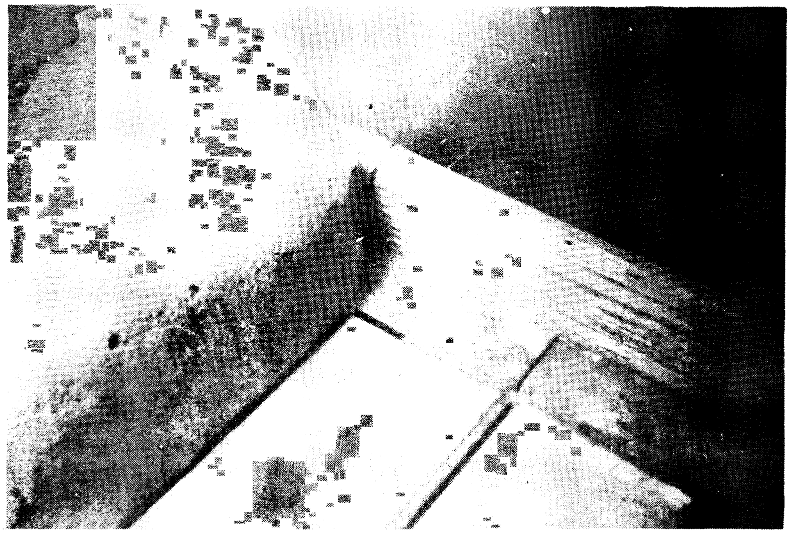


Fig. 8

Erosion of downstream bank after test using sand-cement ratio of 300:1

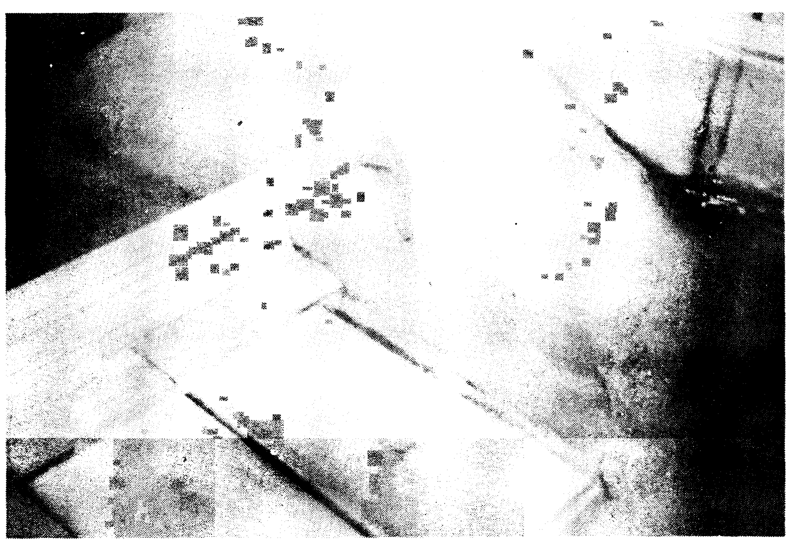


Fig. 9

Erosion of downstream bank after test using sand-cement ratio of 500:1

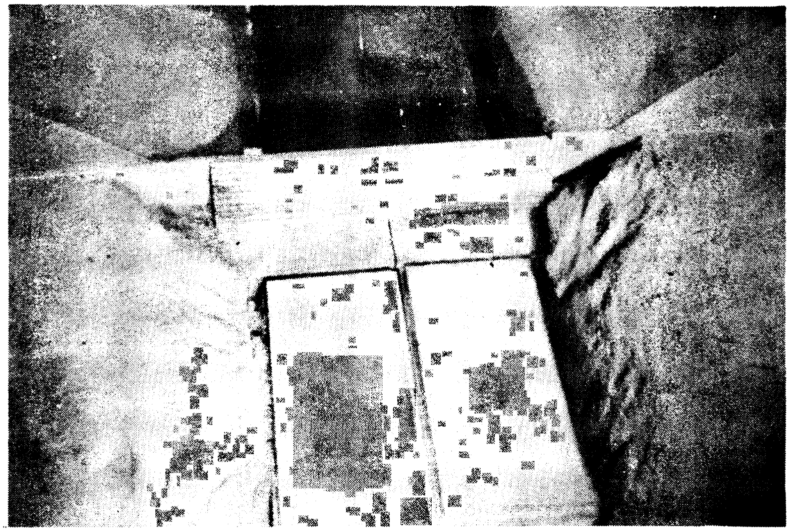


Fig. 10

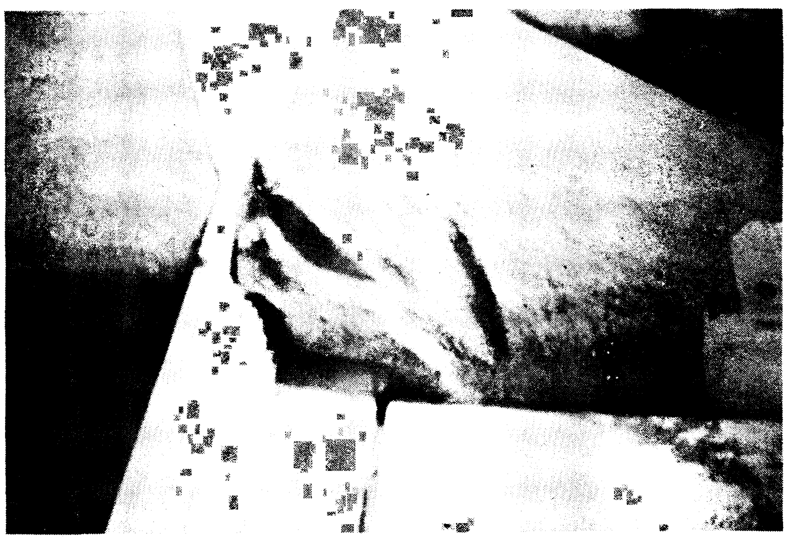


Fig. 11

Erosion of downstream bank after test
using sand-cement ratio of 400:1