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Denver, Colorado, May 22, 1940.

MEMORANDUM TO CHIEF ENGINEER
(D. M. Lancaster)

Subject: A thesis for submission to the graduate school of the University of Colorado in compliance with requirements for the degree of Master of Science.

1. In compliance with the requirements of study for the degree of Master of Science in Civil Engineering in the graduate school of the University of Colorado, I have prepared a thesis entitled, "Experimental Study of the Hydraulic Characteristics in the River Outlets for Grand Coulee Dam." The material which has been deduced from experimental studies made in the Bureau of Reclamation laboratories will subsequently be included in the final reports on the Columbia Basin Project.

2. In obedience with office memorandum No. 186, dated March 2, 1939, the semi-final draft of the thesis is submitted for your approval and release.

Att.

Approved
[redacted]
5-23-1940

[redacted]

Hyd. 83

[illegible] STUDY OF THE HYDRAULIC CHARACTERISTICS IN THE RIVER OUTLETS FOR GRAND COULEE DAM

by

[illegible] Lancaster

B. S. C. E., Colorado State College of

Agriculture and Mechanic Arts, [illegible]

A Thesis submitted to the Faculty of the Graduate School of the University of Colorado in partial fulfillment of the requirements for the Degree

Master of Science

Department of Civil Engineering

1940

This Thesis for the M. S. degree, by

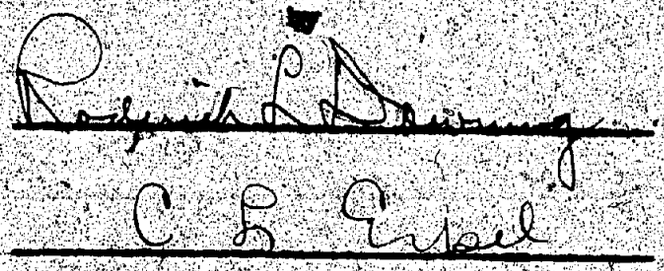
Dale Hatton Lancaster

not proof-read, has been approved for the

Department of

Civil Engineering

by



Date May 31 1940

FOREWORD

The hydraulic experimental program represented in this thesis was conducted in the hydraulic laboratories of the Bureau of Reclamation of the United States Department of the Interior in Denver and Fort Collins, Colorado, under the direct supervision of J. E. Warnock, Engineer, to whom the author is deeply indebted for his untiring efforts and interest which made possible the extensive treatment given this subject.

Credit is also extended to Associate Engineers C.W.Thomas, J.W.Ball, and J.N.Bradley, and Assistant Engineer R.A.Goodpasture for their role in the extensive program of laboratory studies propounded in this dissertation.

The hydraulic structures laboratory in which these studies were made functions as a section of the Materials, Testing and Control Division of the Denver office of the Bureau of Reclamation under the general supervision of R.F.Blanks, Engineer and Arthur Ruettgers, Senior Engineer. All general engineering and designing is under the supervision of W.H. Nalder, Assistant Chief Designing Engineer, and J. L. Savage, Chief Designing Engineer. All engineering and construction is under the direction of S. O. Harper, Assistant Chief Engineer, and R.F.Walter, Chief Engineer. All activities of the Bureau of Reclamation are supervised by John C. Page, Commissioner, whose office is in Washington, D.C.

The author wishes to express his appreciation to the Bureau in granting him permission to use the material contained in this thesis. Responsibility for all statements and conclusions presented are assumed by the author as entirely personal.

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I. Introduction

Purpose of Outlets

As a general rule, in dams constructed for the purpose of flood control, power development, or irrigation, provision must be made for the release of water when the elevation of the reservoir surface is below that of the spillway crest. This objective may be accomplished by one or more of the following methods:

1. Outlets through the spillway section.
2. Outlets through the abutment section of the dam, or
3. Release of water by power development.

Except in a very few instances where the river channel is composed of good rock, method 2 requires a separate outlet. Since this outlet both usually increases the cost of the structure, the majority of the large dams designed in recent years by the Bureau of Reclamation have employed method 1, or, where a power plant is included in the project, a combination of methods 1 and 3. The quantity of water released in the production of power is small in comparison to the amount required for irrigation or flood control, and is a function of the power load, hence has little relation to the river demand.

The release of water at any reservoir elevation is necessitated to prevent infringement of prior rights which may exist downstream, to evacuate storage in anticipation of a flood, to prevent interference with fish migration, or to furnish irrigation water downstream. These purposes are also used to divert the flow of the river during the construction period.

The passages mentioned in 1 and 2 are termed river, or flood control outlets. This paper is limited to the study of high-pressure outlets or those having an effective head of more than 75 feet, which pass through the spillway section of overflow dams.

It is deemed advisable to distinguish between outlets and sluiceways. The distinguishing factor lies in the purpose for which the passages are used. The purposes of outlets have been given above, whereas the sluiceways are primarily to remove accumulated sand or silt from the bottom of the reservoir. Although sluiceways are sometimes employed to control the reservoir elevation, the term is reserved for passages whose function is to keep the reservoir free of silt deposits.

The maximum allowable head on an outlet controlled by a slide gate is necessarily limited for structural reasons. To minimize the head on the gates, the outlets are placed in tiers at various elevations, the number in each tier being controlled by the river demand. In the program of operation, any two adjacent tiers may be operated simultaneously.

If a power plant is incorporated in the project, the lower tier of outlets is usually placed at a relatively high elevation since the reservoir cannot be drawn below a minimum head required for the development of power. Contrarily, if no power plant exists, the outlets are placed at a lower elevation so that the full storage capacity of the reservoir may be utilized.

History of River Outlets

Many of the outlet works which have been in operation for several years have given a considerable amount of trouble. In

way where the valves were placed near the upstream end of the passage and a region of sub-atmospheric pressure, conducive to cavitation, was formed in the gate chamber.

In order to fully appreciate the effect of cavitation, the results of the past experiences at the Roosevelt Dam, and others, should be brought to mind[1]. During the construction of this dam, the river was diverted through a tunnel controlled by three Stoney gates. After the tunnel had been in operation for some time an inspection revealed that the roller trains of the gates were either broken or entirely gone, the gate seats were badly damaged, and bolts in the assembly had been loosened by vibration. The concrete lining of the tunnel had disintegrated, a hole 20 feet in diameter being formed in the crown of the tunnel. Two hundred and fifty cubic yards of concrete were required to make the necessary repairs to the conduit.

The entire tunnel was then lined with steel plates to a point 25 feet downstream from the gates. After the tunnel had been operated for some time under a maximum head of 280 feet, it was observed that the steel lining had been torn away and the concrete beneath the steel plates again damaged. Conduits controlled by bronze slide gates were placed in the tunnel to divert the water in the future.

At Shoshone Dam a similar condition existed. The lower outlet tunnel which was controlled by three slide gates operating under a maximum head of 85 feet received considerable damage. Balanced needle valves of the type used at Arrowrock Dam were later installed to remedy the trouble.

The north tunnel of the Bathfinder Dam contained four slide gates, each controlling a 76 1/4-inch by 44-inch opening. Piers separated the gates, and the gate chamber was lined with 1/2-inch steel plates. After a short period of operation it was necessary to replace the steel linings with ribbed cast-iron linings. During the operation at a discharge of 2,200 second-feet a reverberating sound was quite audible. The floor slab in the lower shaft was deflected 3 1/2 inches due to the partial vacuum being formed in the gate chamber. An inspection revealed that one of the cast-iron linings had been torn out necessitating repairs to the tunnel. A vent was provided in the roof immediately below the gates to give sufficient air supply. Damage still occurred to the gates and, finally, they were abandoned in favor of the balanced needle valves. As the discharge through this type of valve is less than that through a slide gate, it became necessary to drive another tunnel through the south side of the dam in order to pass the desired amount of water. Although balanced needle valves were used in this tunnel, the required amount of air was greater than originally planned.

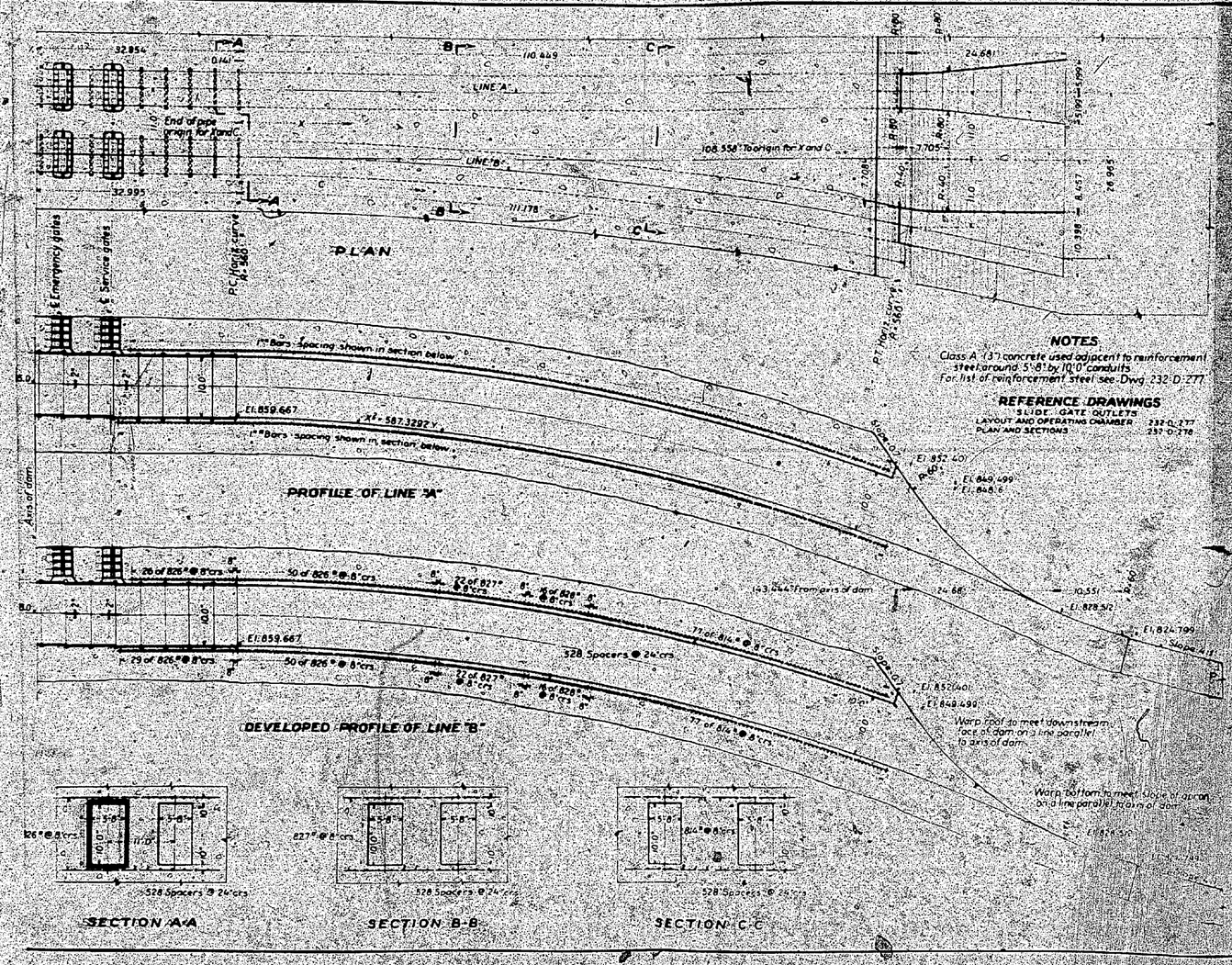
Similar experiences have been encountered at Strawberry, Arrowrock, and Elephant Butte Dams.

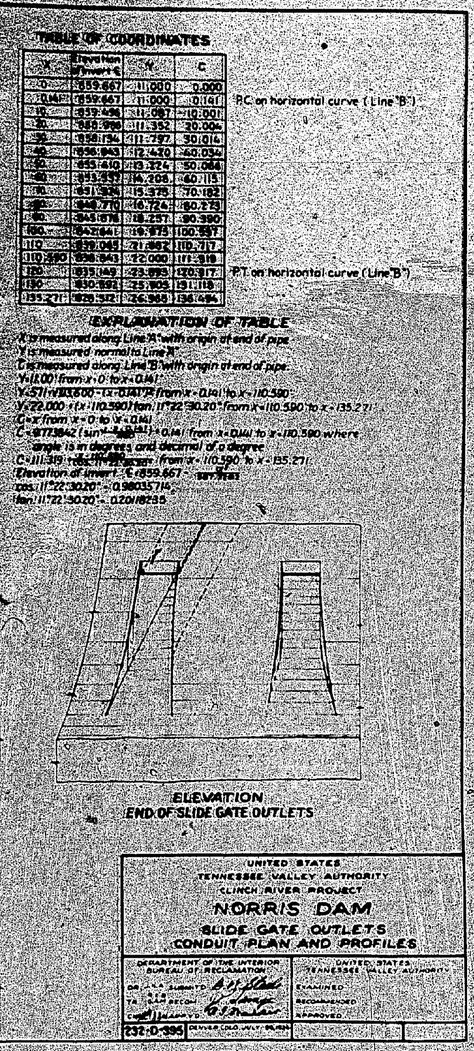
Outlets of the type discussed in this paper were first studied by the Fort Collins hydraulic laboratory of the Bureau of Reclamation in connection with the design for the Norris Dam[2]. In the initial design, the inverts of the outlets were horizontal: the issuing jets impinged on the tailwater causing eddy currents which carried loose material onto the apron. The resulting erosion of the concrete forced abandonment of this design.

In an effort to cause the jets from the passages to discharge nearly parallel to the sloping apron, the profile of each conduit was shaped on a parabola with the origin at the control gate. The advantage of discharging the jets into the stilling basin was to form a hydraulic jump, thus dissipating the energy before the flow reached the unpaved river section. Since the outlets discharged at a low elevation relative to the tailwater, the jets had no chance of spreading by natural means, consequently, the velocity distribution was so uneven in the stilling basin that an efficient hydraulic jump failed to form. The condition was improved by diverging one outlet in each pair forming an angle of 11 degrees with the straight one. This arrangement showed an improvement but satisfactory conditions of flow were not obtained until the openings in the face of the dam were flared as shown on figure 1. Laboratory facilities at the time these studies were made did not permit the study of pressures throughout the length of the conduit.

Summary

In the course of the test program extending over a period of several years, the design of conduit entrances has been explored extensively, resulting in a bellmouth entrance not subject to intensive sub-atmospheric pressures conducive to cavitation, but at the same time structurally simple; the hydraulic behavior of slide gates has been analyzed by the use of detailed models. The possible erratic performance of outlets parabolic in profile has been brought to light and that type of design discarded as a result; a new design of outlets incorporating an elbow and cone at the downstream end has been developed which is conducive to the minimizing of spray conditions





NOTES

Class A (3") concrete used adjacent to reinforcement steel around 5'-8" by 10'0" conduits
For list of reinforcement steel see Dwg. 232-D-277

REFERENCE DRAWINGS

SLIDE GATE OUTLETS
LAYOUT AND OPERATING CHAMBER — 232-D-277
PLAN AND SECTIONS — 237-D-178

TABLE OF COORDINATES

X	Elevation Invert ℄	Y	C	
0	859.667	11.000	0.000	
0.041	859.667	11.000	0.041	P.C. on horizontal curve ("Line B")
10	859.496	11.097	10.000	
20	859.198	11.352	20.004	
30	858.794	11.797	30.016	
40	858.343	12.420	40.034	
50	857.910	13.224	50.044	
60	857.557	14.206	60.115	
70	857.324	15.375	70.182	
80	857.270	16.724	80.275	
90	857.474	18.257	90.390	
100	857.841	19.975	100.537	
110	858.445	21.892	110.717	
110.590	858.943	22.000	111.318	P.T. on horizontal curve ("Line B")
120	859.149	23.895	120.917	
130	859.894	25.905	131.118	
135.271	860.312	26.968	136.494	

EXPLANATION OF TABLE

X is measured along "Line A" with origin at end of pipe
Y is measured normal to "Line A"
C is measured along "Line B" with origin at end of pipe
Y = 11.000 from X = 0 to X = 0.041
Y = 5.711 + 10.000 - (+ 0.041)*2 from X = 0.041 to X = 110.590
Y = 22.000 + (X - 110.590) tan 11°22'30.20" from X = 110.590 to X = 135.271
C = X from X = 0 to X = 0.041
C = 8.773842 (sin 4 θ - 4 θ/81) + 0.041 from X = 0.041 to X = 110.590 where
θ = angle in degrees and decimal of a degree
C = 111.309 - (110.590 - X) cos 11°22'30.20" from X = 110.590 to X = 135.271
Elevation of invert ℄ = 859.667 - 5.711 X
cos 11°22'30.20" = 0.980357114
tan 11°22'30.20" = 0.201182135

UNITED STATES
TENNESSEE VALLEY AUTHORITY
CLINCH RIVER PROJECT

NORRIS DAM
SLIDE GATE OUTLETS
CONDUIT PLAN AND PROFILES

DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	UNITED STATES TENNESSEE VALLEY AUTHORITY
DES. & DRAWN	EXAMINED
TR. & LAY. SECTION	RECOMMENDED
	APPROVED
232-D-395	REVISED (ADD ALT. SLIDE GATE OUTLETS)

FIGURE 1. NORRIS DAM OUTLETS

on the downstream
energy in the
thus minimizing
the outlet exit
interference wi

Although
Grand Coulee Da
same type of ou
River in Califor
projects. The
which might hav
installations
earlier date.

on the downstream face of the spillway, and to the dissipation of energy in the outlet flow in the same manner as that down the spillway thus minimizing erosion of the river bed. The structural features of the outlet exits have been so developed as to cause only slight interference with the flow down the spillway face.

Although the original studies were for the purpose of designing Grand Coulee Dam in the Columbia River in the State of Washington, the same type of outlet is being installed in Shasta Dam in the Sacramento River in California,[3] and, without doubt, will be used in future projects. The studies as a whole have demonstrated improvements which might have been incorporated in other dams with similar installations had the laboratory experiments been started at an earlier date.

II - INITIAL STUDIES OF THE GRAND COULEE OUTLETS

Outlets with Horizontal Profile

The Grand Coulee Dam (Figure 2) is being constructed by the Bureau of Reclamation to furnish irrigation water and to produce power for the adjacent territory.[4] Preliminary investigations showed that ten pairs of outlets at elevation 954.67 and ten pairs at elevation 1108.67 were necessary to meet the river demand[5] (Figure 3).

The initial studies of the river outlets were made on a model scale of 1:120 at the Fort Collins laboratory of the Bureau of Reclamation. In these early investigations the only objective was to minimize the erosion of the river bed caused by the issuing jets; no consideration was given to the condition within the conduits.

Erosion Caused by Operation of Outlets with Horizontal Profile

In the original design, the outlets were horizontal, as in the initial design of the Norris Dam, with a cross section of 5 feet, 8 inches by 10 feet high. The jets issuing from the lower tier impinged on the tailwater downstream from the bucket resulting in severe splash and erosion of the river channel to bedrock in this region (figure 4). A similar result was obtained when the upper tier was operated; a greater area of bedrock was exposed than in the previous case (figure 5). It was on the basis of these investigations that the outlets having a horizontal invert were abandoned.

The rectangular cross section was abandoned in favor of a circular one because the periphery of a circular cross section of given area is less than that for a rectangle of the same area, the strength of a circular section is greater than that of a straight

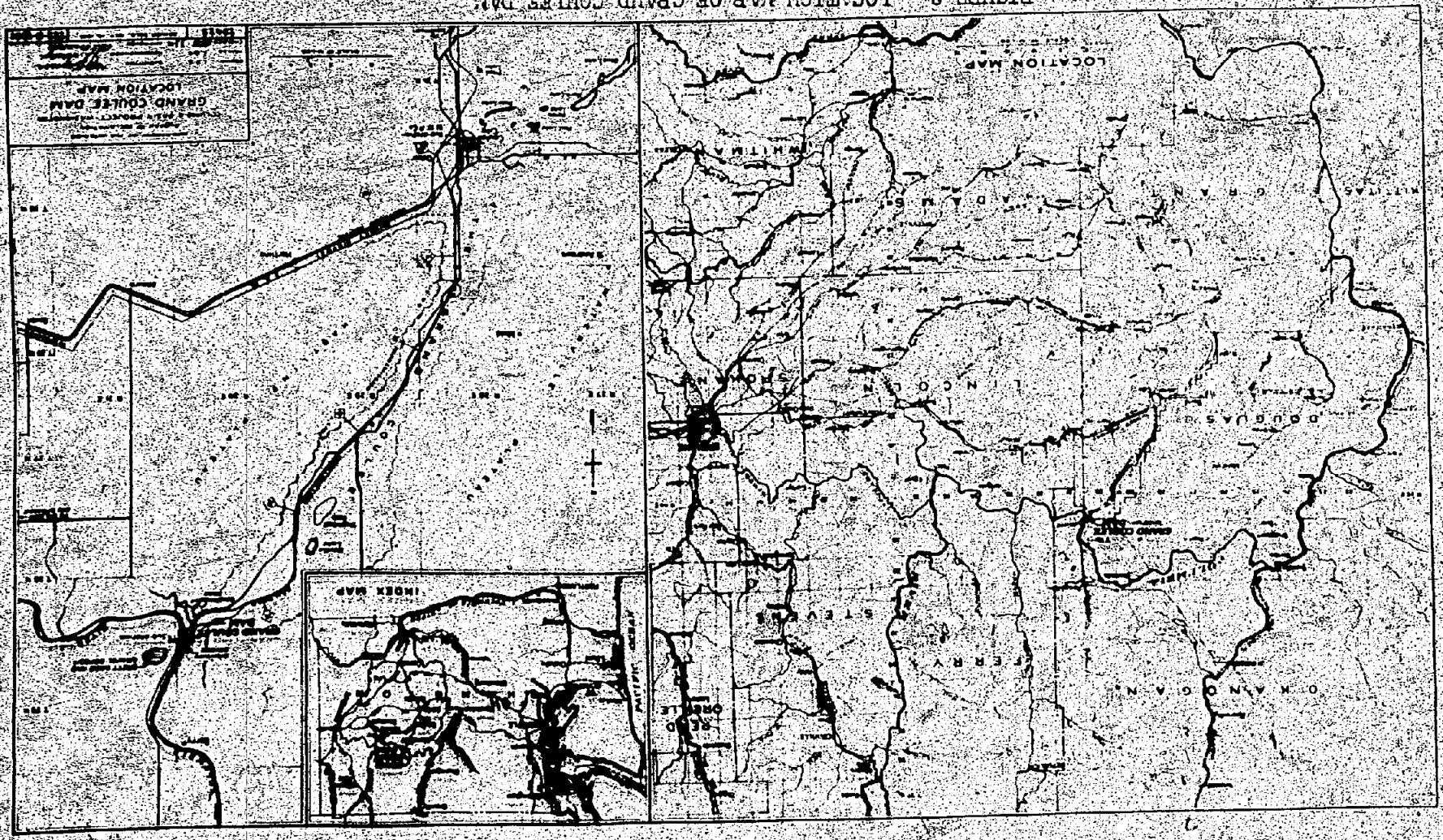
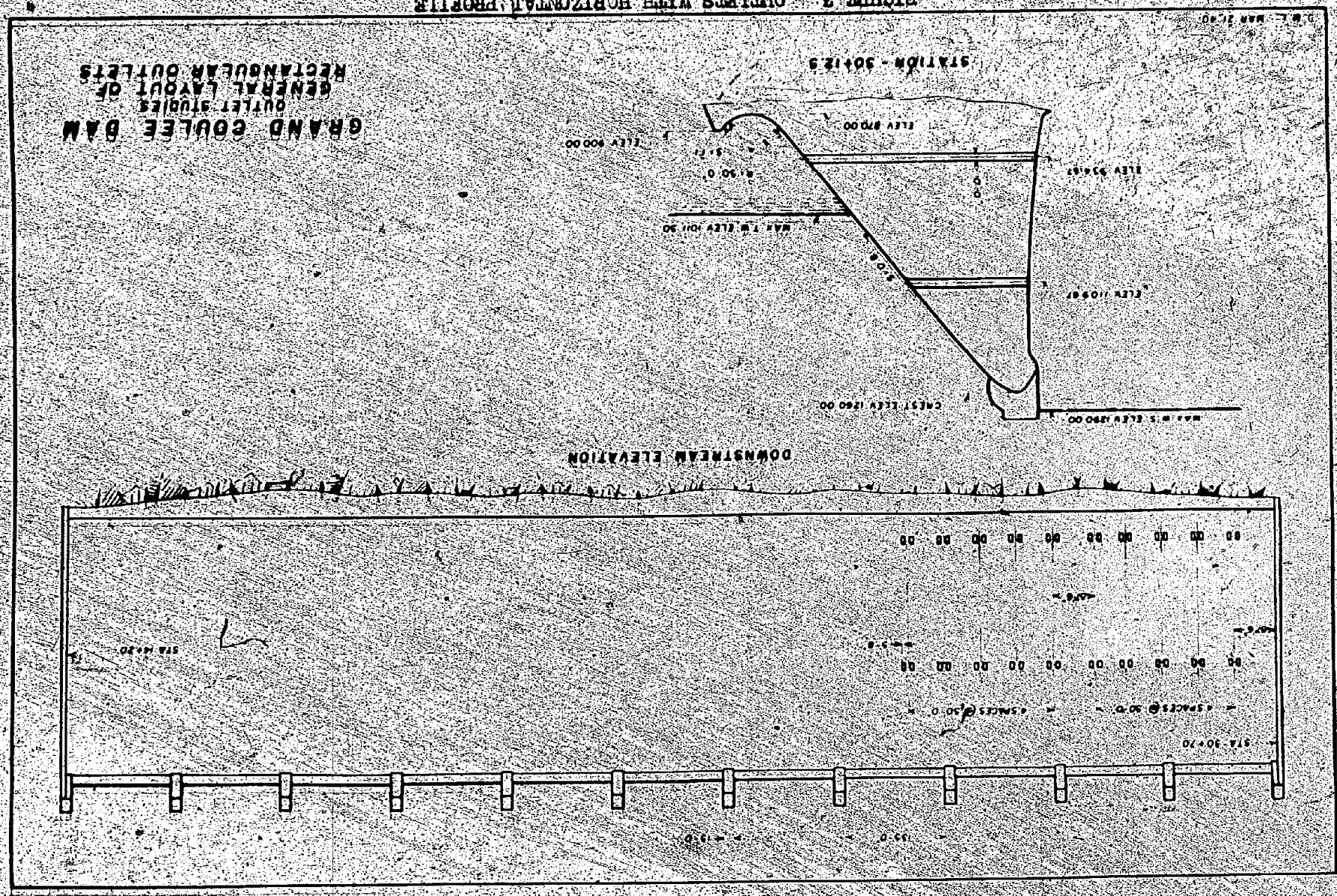
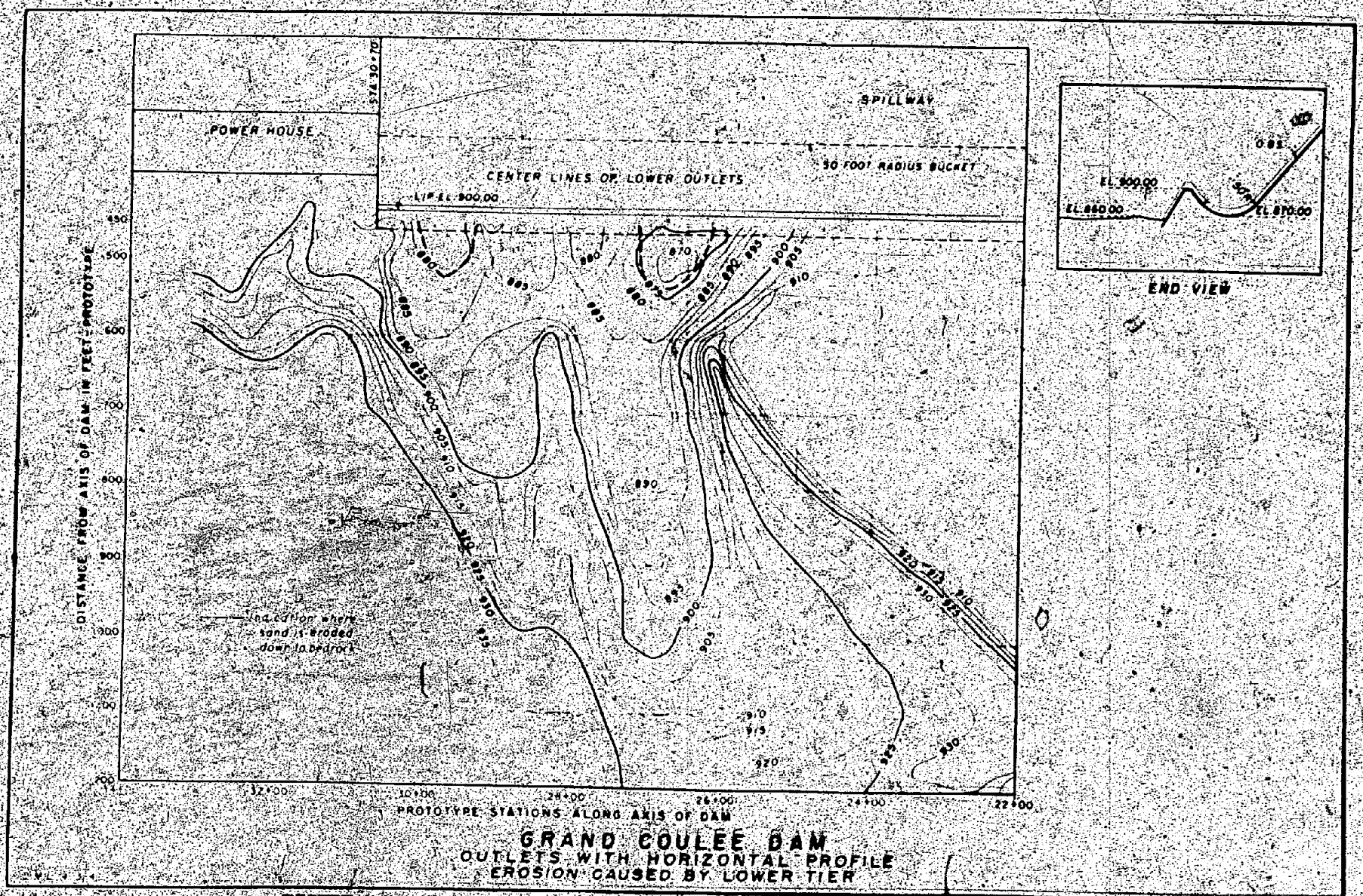


FIGURE 2. LOCATION MAP OF GRAND COULEE DAM.

FIGURE 3. OUTLETS WITH HORIZONTAL PROFILE





GRAND COULEE DAM
OUTLETS WITH HORIZONTAL PROFILE
EROSION CAUSED BY LOWER TIER

FIGURE 4. EROSION CAUSED BY LOWER OUTLETS WITH HORIZONTAL PROFILE

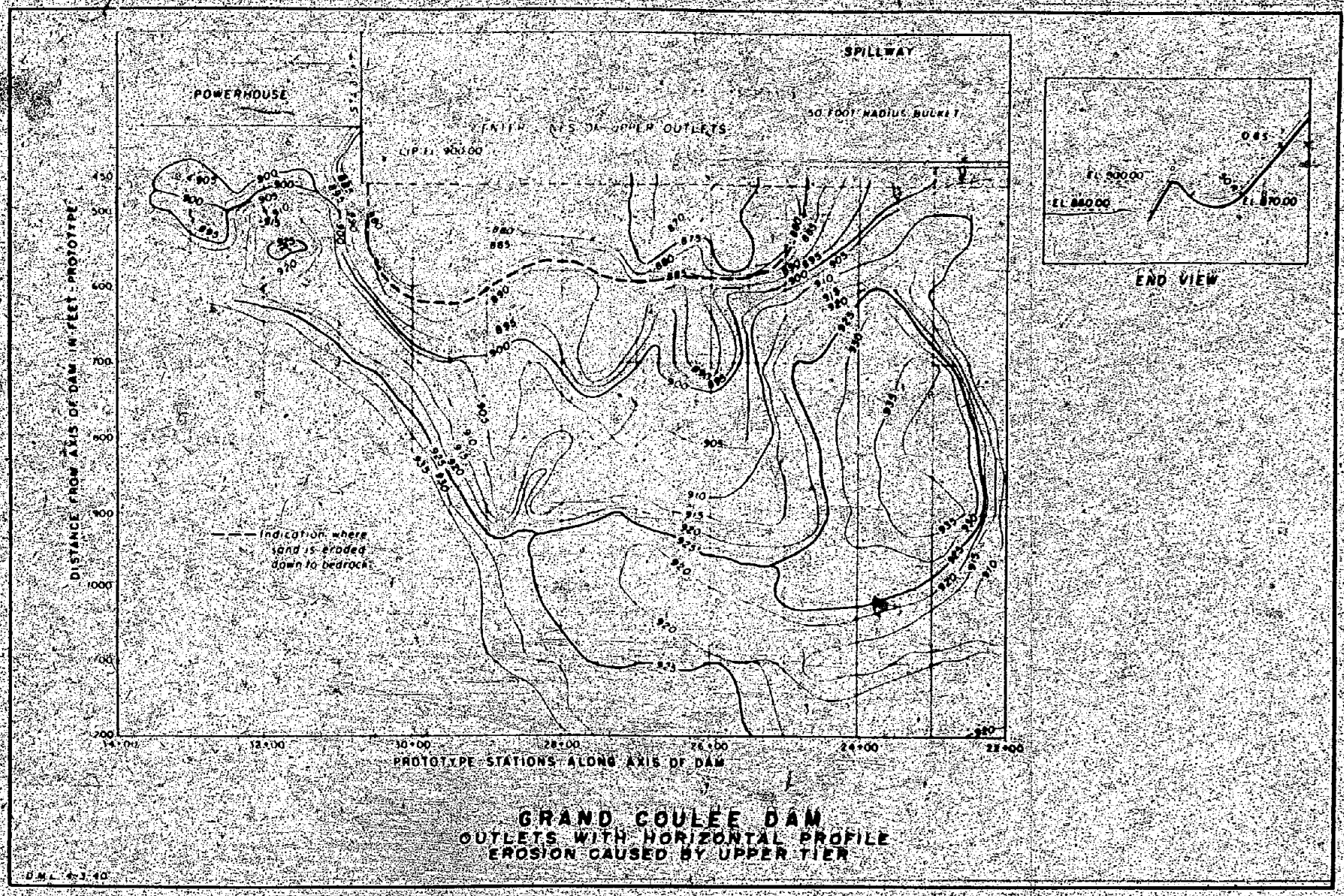


FIGURE 5. EROSION CAUSED BY UPPER OUTLETS WITH HORIZONTAL PROFILE

[illegible], and the construction difficulties involved in the case of the circular section are less.

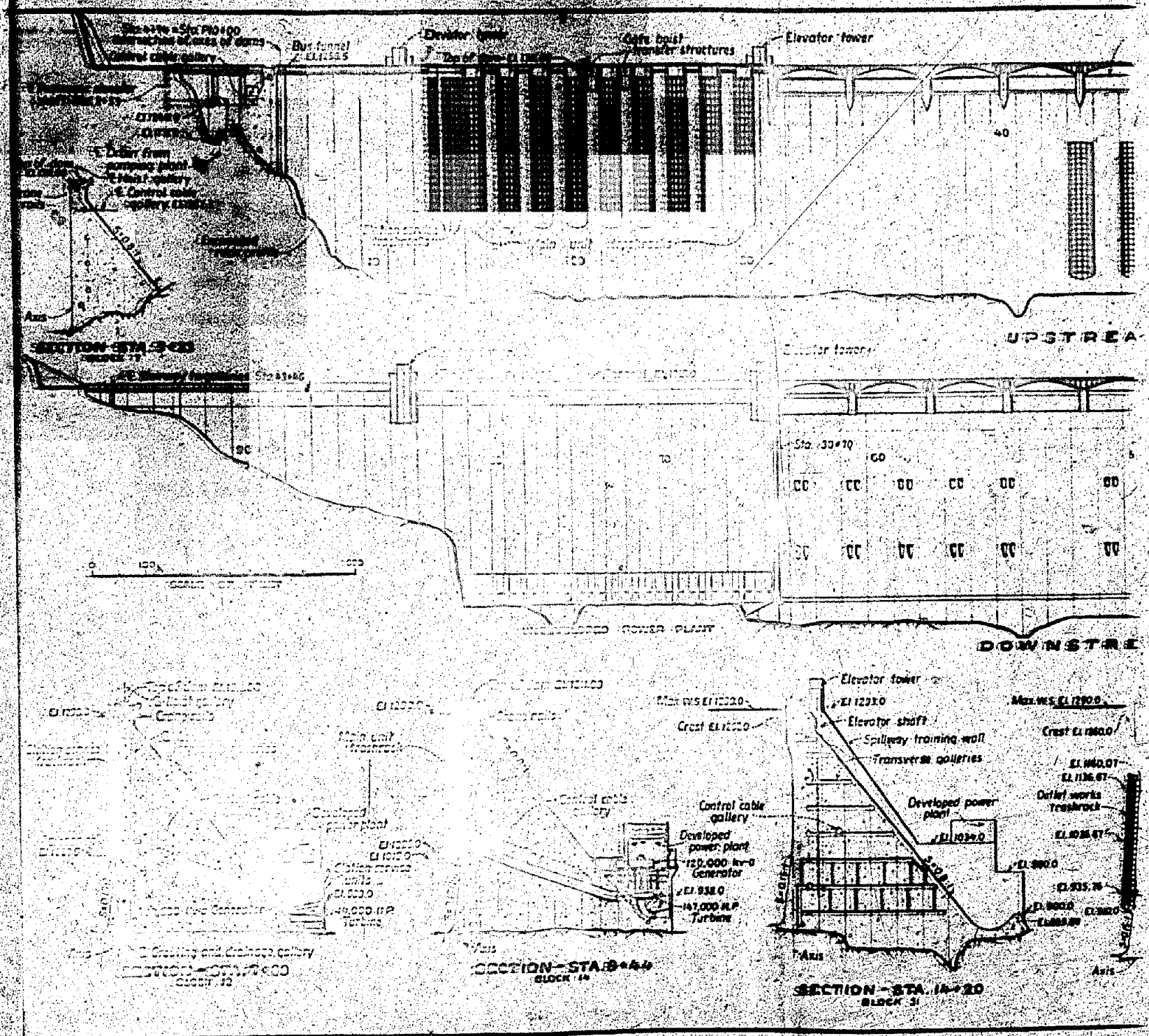
[illegible]

The objective was to shape the passages through the dam so that the leaving jets would impinge within the bucket which was developed to dissipate the energy of the spillway flow (Figure 6). This would cause the energy to be dissipated before the flow reached the unprotected part of the river bed.[6] Since the trajectory of a jet discharging freely from an orifice follows a parabola it seemed logical to place the profile of each outlet on a parabolic curve.

Further study of the requirements of stream flow below the dam made it necessary to increase the number of outlets to sixty. This required an entirely new arrangement of the conduits; ten pairs were placed at elevation [illegible], an equal number at elevation [illegible], and a third tier at elevation [illegible] as shown on section at station [illegible] on figure 6. On the [illegible] model only one pair of the modified outlets were installed in each tier. At the time these early studies were made the maximum allowable head on the control gates was not known, hence each tier was operated at full reservoir elevation.

[illegible]

The lower tier of outlets was placed with the downstream end of the parabola tangent to the bucket which established the origin at some distance from the control gate. The most severe erosion occurred with low tailwater which would be the condition when the conduits were first opened. With one or two outlets discharging, erosion extended to bedrock at the sides of the fan-shaped hull. The difference in



Bus tunnel
Elevator tower
Gate hoist
transfer structures
Elevator tower
Control cable gallery
Drain from pumping plant
Hoist gallery
Control cable gallery
Axis
SECTION-STA.3+23
UPSTREA
Elevator tower
Sta. 30+70
DOWNSTRE
Elevator tower
Elevator shaft
Spillway training wall
Transverse galleries
Max. W.S. El. 1290.0
Crest El. 1260.0
Control cable gallery
Developed power plant
Outlet works trashrack
Developed power plant
120,000 kv-a Generator
147,000 H.P. Turbine
Axis
SECTION-STA.8+44
BLOCK 14
SECTION-STA.14+20
BLOCK 31

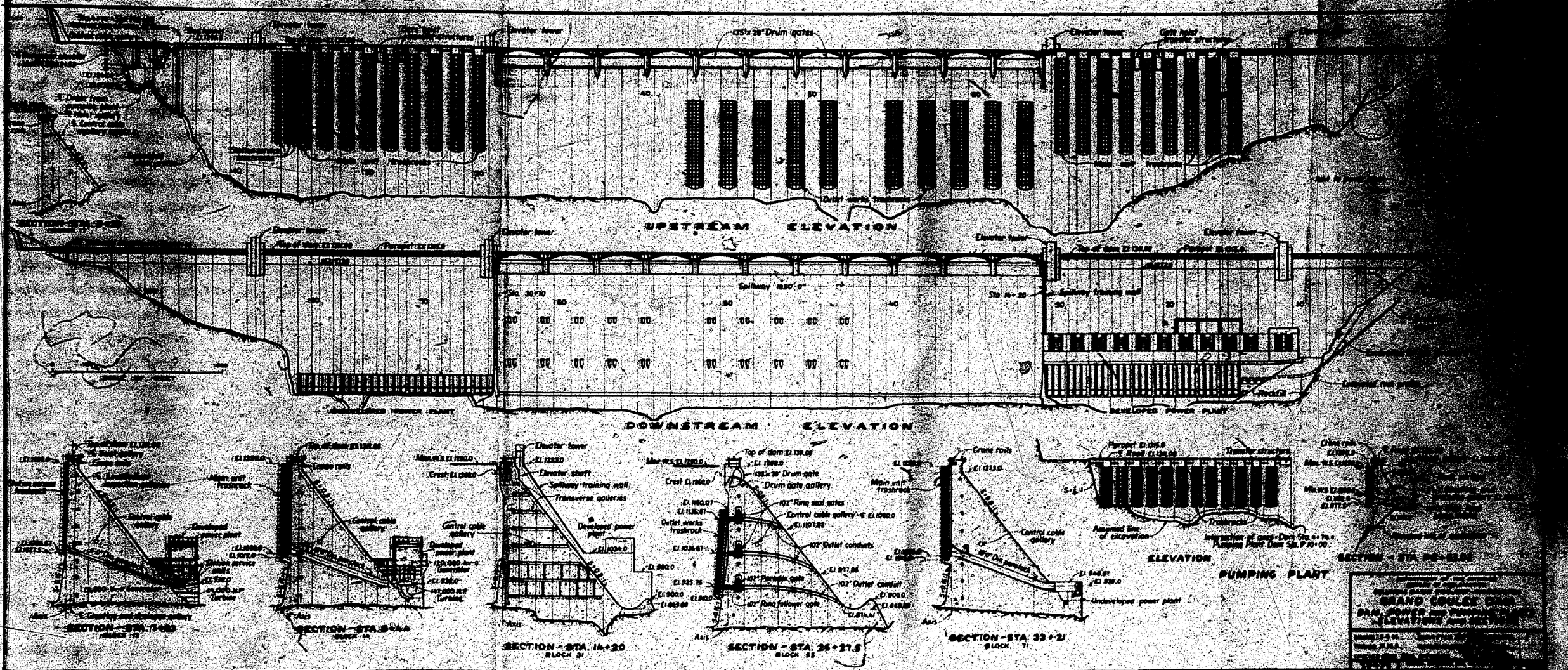
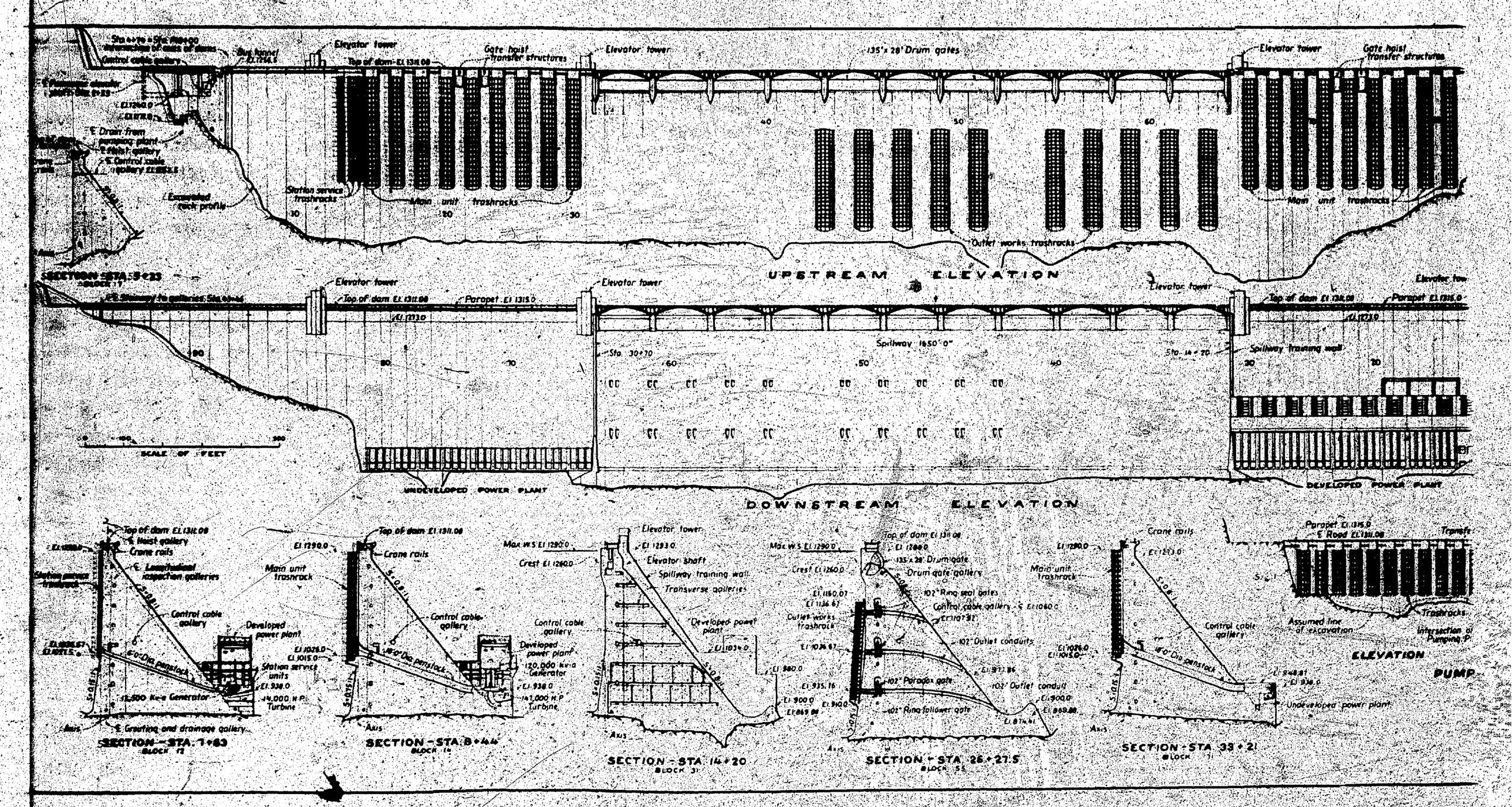


FIGURE 6. OUTLINE AND GALLERY ARRANGEMENT



Elevator tower
Gate hoist transfer structures
135' x 28' Drum gates
Top of dam El. 1311.08
Bus tunnel
Control cable gallery
Drain from pumping plant
Hoist gallery
Control cable gallery
Excavated rock profile
Station service trashracks
Main unit trashracks
Outlet works trashracks
UPSTREAM ELEVATION
SECTION - STA. 5+23
Parapet El. 1315.0
Spillway 1650' 0"
Sta. 30+70
Sta. 14+20
Spillway training wall
SCALE OF FEET
UNDEVELOPED POWER PLANT
DEVELOPED POWER PLANT
DOWNSTREAM ELEVATION
Crane rails
Longitudinal inspection galleries
Control cable gallery
Developed power plant
Station service units
Grouting and drainage gallery
SECTION - STA. 7+63
BLOCK 12
Main unit trashrack
SECTION - STA. 8+44
BLOCK 14
Elevator shaft
Spillway training wall
Transverse galleries
Developed power plant
SECTION - STA. 14+20
BLOCK 31
Drum gate gallery
102" Ring seal gates
102" Outlet conduits
102" Paradox gate
102" Ring follower gate
SECTION - STA. 26+27.5
BLOCK 55
Assumed line of excavation
Trashracks
Intersection of Pumping P
ELEVATION
PUMP
Undeveloped power plant
SECTION - STA. 33+21
BLOCK 71

UPSTREAM ELEVATION

DOWNSTREAM ELEVATION

DEVELOPED POWER PLANT

SECTION - STA. 14+20
BLOCK 31

SECTION - STA. 26+27.5
BLOCK 65

SECTION - STA 33+21
BLOCK 71

ELEVATION
PUMPING PLANT

SECTION - STA P6+63.02

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
DAM, POWER AND PUMPING PLANTS
ELEVATIONS AND SECTIONS

FIGURE 6. OUTLETS WITH PARABOLIC

[illegible] from the [illegible] river channel caused a larger area of bedrock to be exposed on the left side.

Less erosion occurred when the jets were spread by diverging the [illegible] (Figure 7). It should be remembered that the indicated erosion is the result of an abnormal condition as the tailwater was low and the reservoir elevation was a maximum. This established the shape of the lower [illegible] downstream from the control gates (Figure 8); later [illegible] determined the upstream portion.

Erosion Caused by Operation of Intermediate Tier

As the trajectories of the intermediate tier intersected the tailwater at an acute angle, the deflection of the jets varied with the tailwater elevation, which in turn depends upon the release of water by the outlets, spillway or powerhouse. When the tailwater was above elevation 950, the jets passed beyond the bucket lip causing severe erosion to the river channel. In order to overcome this difficulty, it would have been necessary to make the conduits discharge more nearly parallel to the face of the dam.

Erosion Caused by Operation of Upper Tier

The jets from the upper tier intersected the tailwater nearly at a right angle, consequently, the tailwater elevation had no effect on the deflection of the entering streams. As the jets always impinged within the bucket, little erosion occurred.

Erosion Resulting from Operation of Combinations of Tiers

All combinations involving the operation of the intermediate tier were undesirable when the tailwater was above elevation 950.

RIGHT OUTLET DISCHARGING

PAIR OF OUTLETS DISCHARGING

RESERVOIR ELEVATION 1288.0 TAILWATER ELEVATION 930.0

FIGURE 7. FLOW CONDITIONS AT RIVER BED REGION - FINAL DESIGN LOWER OUTLETS

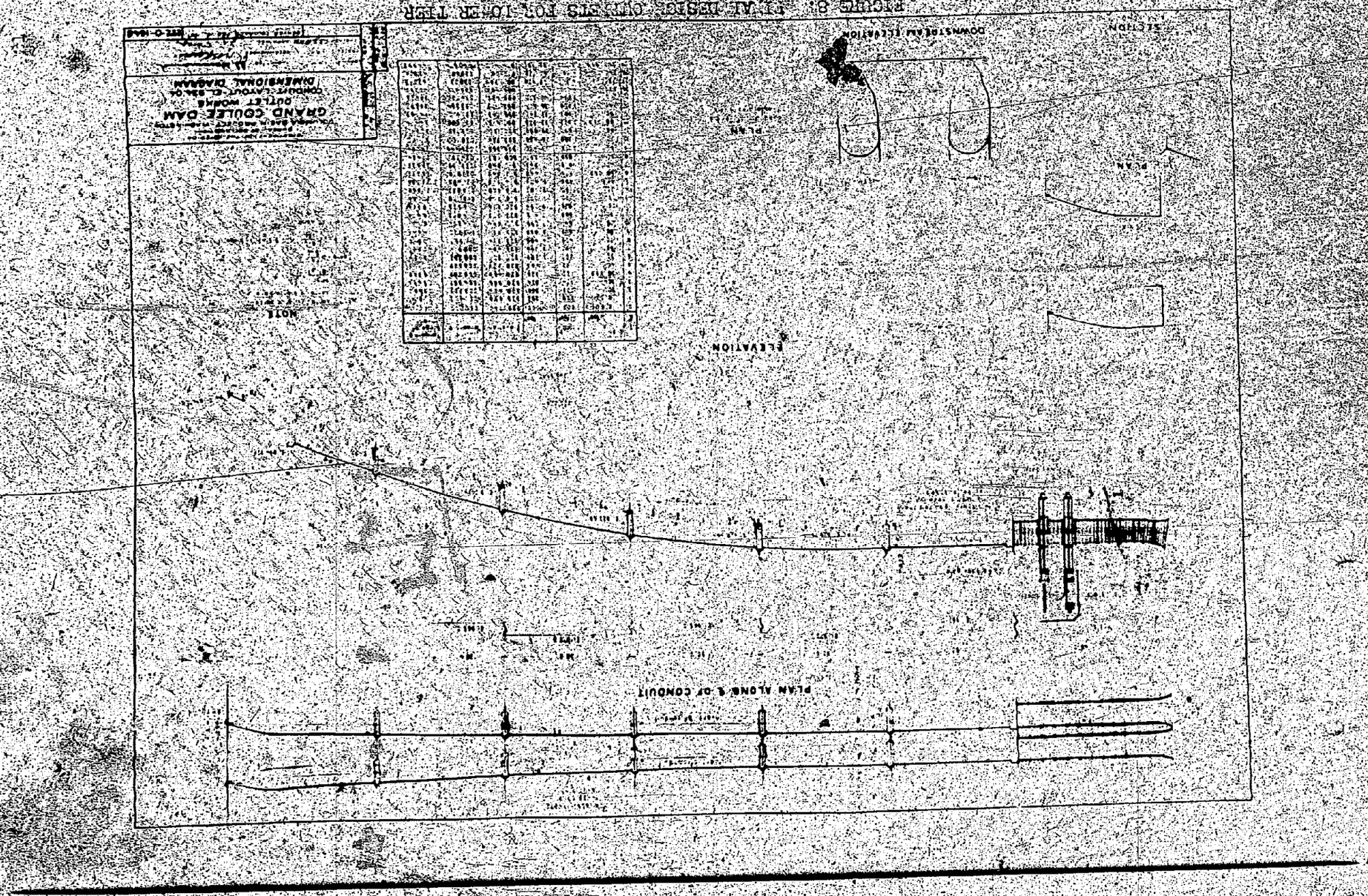


FIGURE 8: FINAL DESIGN OUTLETS FOR LOWER TIER

The simultaneous operation of the upper and lower tiers gave satisfactory results when the tailwater was at elevation 950 or less.

III - [illegible] STUDIES IN OUTLETS WITH PARABOLIC PROFILE

Cavitation of Outlet Entrances at Madden Dam

During this period of study reports came from the Madden Dam in the Chagres River in the Panama Canal Zone[7] concerning the poor performance of the outlet works. Examination of the conduits after a few weeks of operation disclosed destruction of a portion of the roof immediately downstream from the entrances. Since it was localized and definite in its extent, there appeared to be but one answer to the cause, namely, cavitation due to the severe sub-atmospheric pressures. Consensus of opinion was that the destruction was mainly caused by the unsymmetrical entrance resulting from the bottom of the reservoir being too close to the entrances. Piezometers in the conduit indicated sub-atmospheric pressures as low as 29 feet of water.

Professor H. A. Thomas performed some experiments on the bell-mouthed entrances at the laboratory of the Carnegie Institute of Technology.[8] For these studies the model was built in a tank so that the atmospheric pressure could be reduced to correspond to the model scale. The curve forming the flare of the entrance was corrected by the method of trial and error.

Theories of Cavitation

Several theories have been advanced concerning the forces acting during cavitation. Some have explained it by saying that the pressure within the material is sufficiently large to cause destruction when a partial vacuum is formed on the surface. Another theory is that nascent oxygen was released from the air in solution in the low pressure region. Since oxygen is an oxidizing agent, it was assumed

[illegible]

The present accepted theory on cavitation is that cavities are formed in the stream due to the pressure approaching absolute zero; and the collapsing of the cavities as they pass from a region of low to one of high pressure results in damage to the surrounding material. This process begins at a pressure of approximately one-half an atmosphere.[9] Cavitation does not occur in a region where pressure remains low, as shown by J. C. [illegible] at the Massachusetts Institute of Technology.[10]

Design of Bellmouthed Entrance for Grand Coulee Outlets

In order to prevent cavitation in the Grand Coulee outlets, a complete study was made, starting with the bellmouthed entrance, nothing being carried over from previous studies.[11] The Fort Collins hydraulic laboratory studied the entrance under low heads while the Denver laboratory studied the problem under high heads.

Special Equipment Used

Elaborate equipment was constructed for the purpose of obtaining a high head on the hydraulic model. A cylindrical tank 5 feet in diameter and 5 feet long was constructed with a series of concentric distributing cones and a special rack to produce approach conditions similar to those which will exist on the prototype structure. In the downstream end of the tank, the plate holding the orifice was of the floating type which was designed to remain in a plane regardless of any bulging occurring in the tank due to high pressure (figure 9). A 18-inch centrifugal pump operated by a 75-horsepower variable-speed electric motor was purchased to

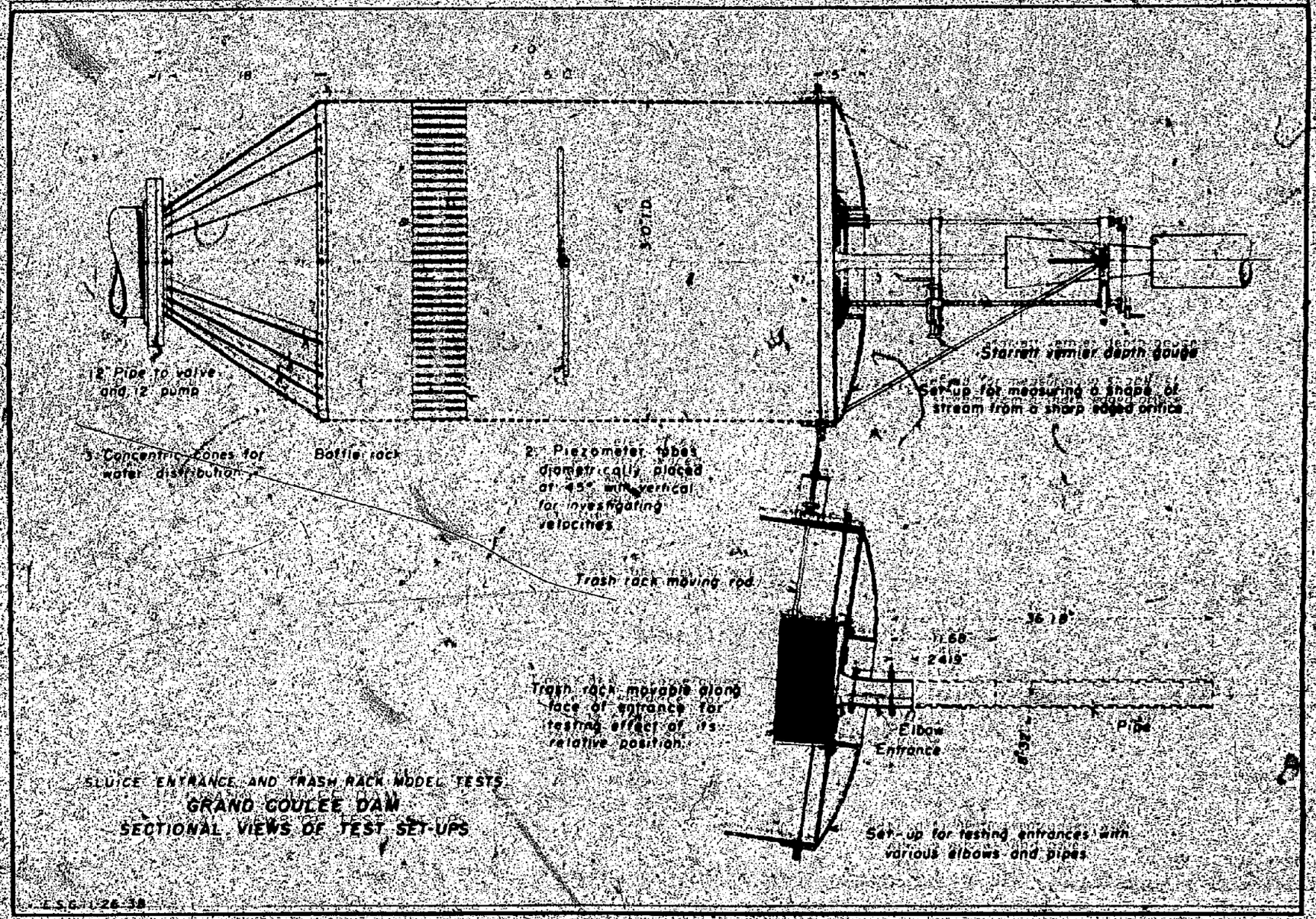


FIGURE 9: ASSEMBLY OF SPECIAL EQUIPMENT

furnish the discharge at the proper pressure head.

A special measuring device to determine the contraction curve of a jet issuing from a sharp-edged orifice was designed. The instrument was capable of measuring the distance along the axis and the diameter of the jet at any section (figure 9).

Analytical Design

The analytical design of the bellmouthed entrance was first studied at the Fort Collins laboratory. The entrance was formed by rotating the ellipse $\frac{X^2}{(1.[illegible])^2} + \frac{Y^2}{([illegible])^2} = 1$ about the axis of symmetry. This entrance, called model "A", was brought to the Denver laboratory for observance under high heads, and low pressures were found to exist (figure 10). Since the upstream face of the dam is on a slope of 0.[illegible], it was necessary to connect the horizontal portion of the passage to the bellmouthed entrance by an 6°-[illegible]' elbow in order to keep the entrance symmetrical. The pressures in the two elbows studied (figure 11) showed that the centrifugal force of the jet as it passed around the bend caused sub-atmospheric pressures on the inner curve.

Laboratory Design

The most certain way to obtain positive pressures in the entrance and elbow was to converge the elbow in the direction of flow. As the minimum diameter of the cone was fixed at 102 inches, it was necessary to increase the diameter of the entrance in order to obtain the convergence. Consequently, a new entrance was determined by measuring the contraction curve of a 3-inch sharp-edged orifice discharging

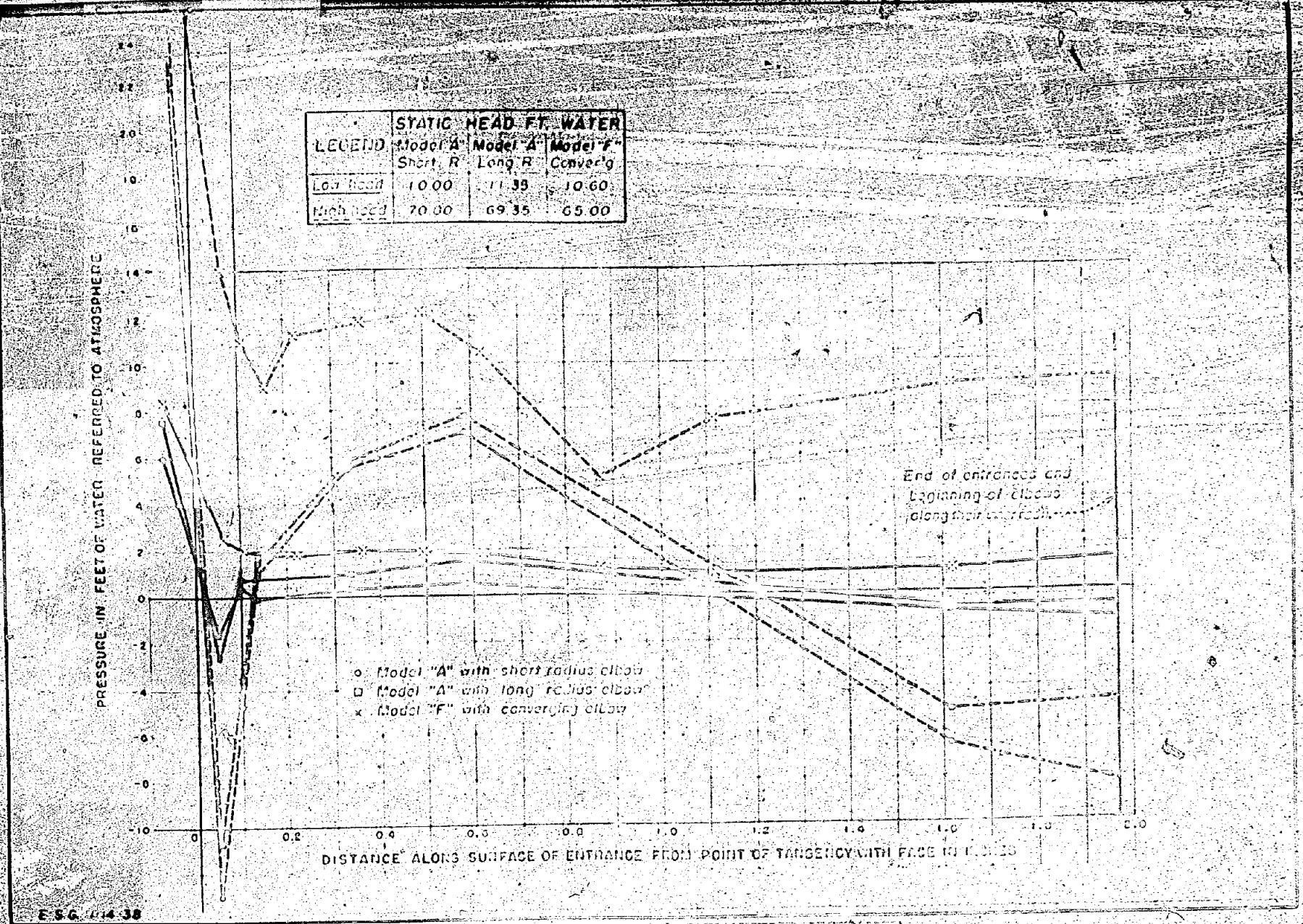


FIGURE 10. PRESSURE DISTRIBUTION FOR DIFFERENT ENTRANCES

PRESSURES AT MIDPOINTS OF ELBOWS

ELBOWS	HEAD IN FEET OF WATER	TOP INNER RADIUS	BOTTOM OUTER RADIUS
Short R with Mod "A" Ent. length 1.75"	HIGH HEAD 70.80	-10.10 *	10.90
	LOW HEAD 10.00	-1.40 *	1.50
Long R with Mod "A" Ent. length 4.16"	HIGH HEAD 69.35	-4.10 *	7.00 *
	LOW HEAD 11.35	-0.60 *	1.25 *

* Obtained by interpolation

LEGEND	STATIC HEAD-FT. Model "A" Short R.	Model "A" Long R.
LOW HEAD	10.00	11.35
HIGH HEAD	70.80	69.35

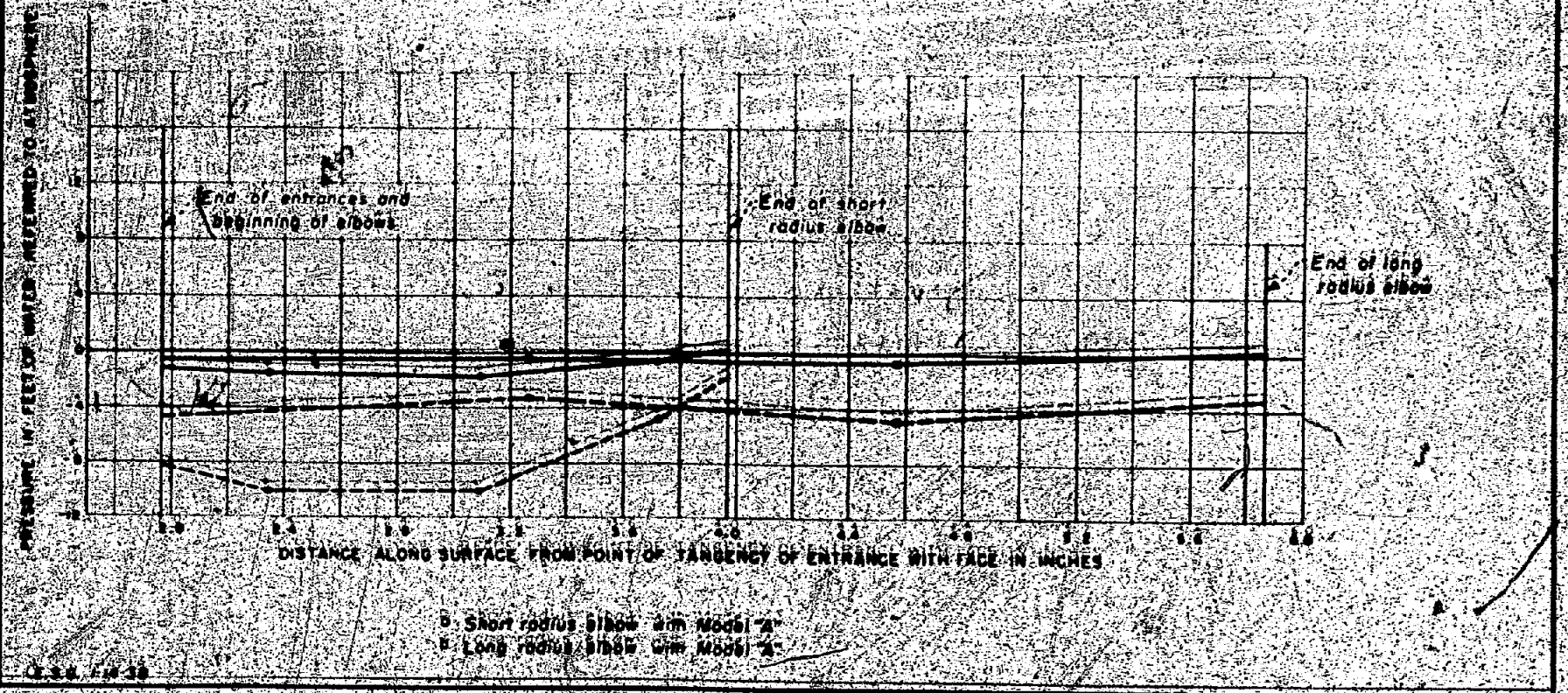


FIGURE 11. PRESSURE DIAGRAM FOR UPSTREAM ELBOWS

freely. The contour of the bellmouthed entrance was then placed on the curve thus obtained. In equation form, the newly developed entrance was represented by $\frac{[illegible]}{([illegible])^2} + \frac{[illegible]}{([illegible])^2} = 1$, which gave diameters slightly smaller than that of the measured jet for all sections upstream from the horizontal tangent. As it was necessary to converge the elbow section, the upstream diameter of the entrance was made of sufficient size to cause a diameter of [illegible] inches to be found at the end of the entrance. The decrease from this diameter to [illegible] inches gave the necessary convergence in the elbow. In order to keep the original length shown in the specifications, it was necessary to include a short length of constant-diameter pipe between the bellmouthed entrance and the elbow (Figure [illegible]). Satisfactory pressures were found throughout the elbow and cone section (Figure 12, model "F"). At the upper tier of outlets the face of the dam is vertical; hence, the converging section was made with a straight centerline.

It should be noted that a factor of safety resulted from neglecting the back pressure due to the frictional losses in the long conduit of the prototype. A length of pipe to insure appreciable frictional resistance was added to the model to show the effect of this back pressure. (Figure 13).

Effect of Placing Outlets in Pairs

In most instances construction economy results by placing two outlets close together in order to be served by one trashrack. At Grand Coulee Dam, the outlets are placed in pairs, 15 feet center to center. To study this condition two of the bellmouthed entrances with converging cones were installed on the pressure tank. The

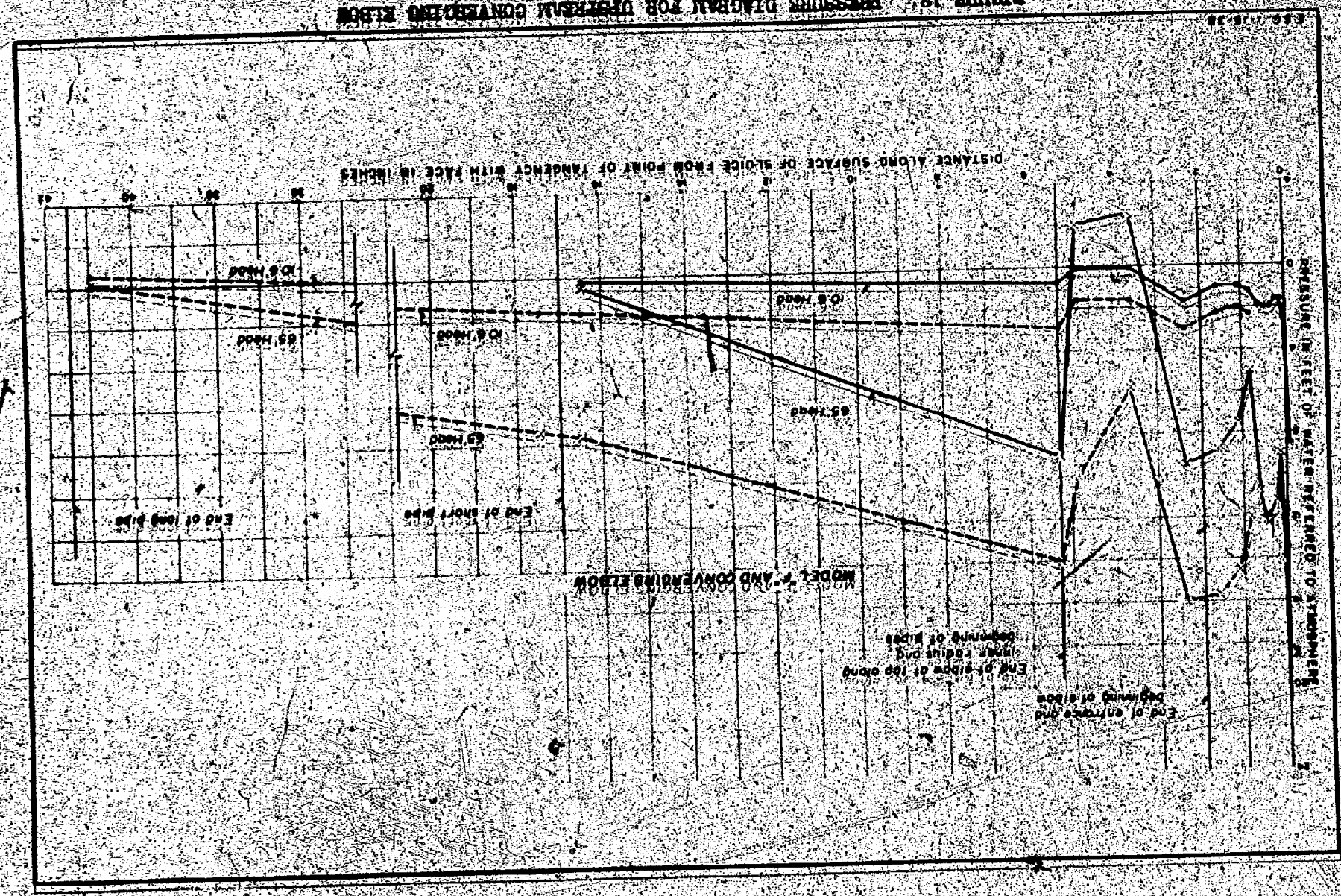


FIGURE 12: PRESSURE DIAGRAM FOR UPSTREAM CONVERGING ELBOW

greatest change in pressure (figure 15) occurred on the side furthest from the axis of symmetry. The pressure decrease here was due to the fact that more contraction existed because of the removal of the balancing force on the opposite side; however, the pressures were considered as satisfactory.

Effect of Trashrack

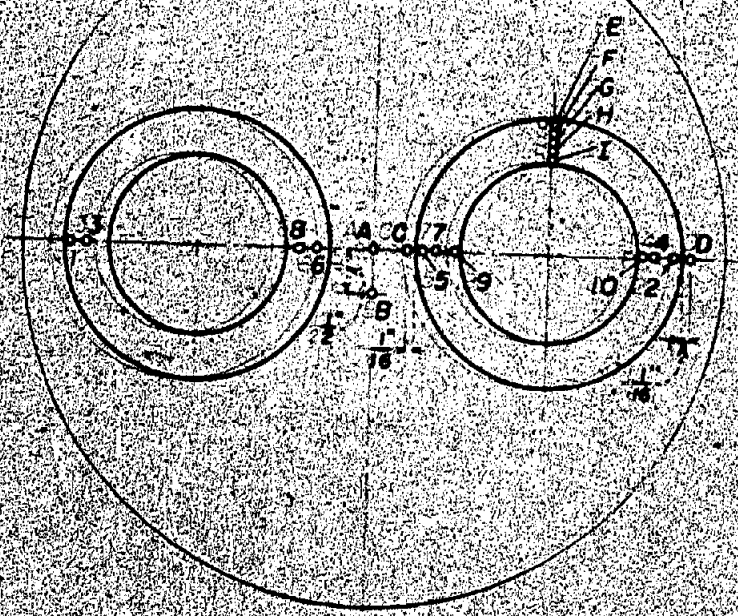
Since the trashrack (figure 9) might affect the entrance conditions, the study would not be complete without considering this factor. Accordingly, the structure was installed in the hydraulic model so that it could be moved vertically through a distance equal to the spacing of the horizontal ribs of the trashrack.

This study revealed that the position of the bottom of the trashrack was the most effective factor, while the effect of rib placement was negligible. A decrease in pressure along the top of the entrance resulted when the bottom of the trashrack was moved toward the spacing. The chosen position gave positive pressures in the entrance (figure 15) and the velocity distribution was such that no trouble should occur downstream.

Additional Usage of High-Pressure Equipment

Prior to the acquisition of the equipment to study the bellmouthed entrance, the hydraulic investigation of gates was not feasible because of the lack of high-pressure pumping equipment. Due to the importance of developing a design to prevent repetition of the conditions at Madden Dam, this high-pressure equipment was purchased and installed. During the course of the entrance studies, it became obvious that it

FIGURE 12. PRESSURE DIAGRAM FOR UPSTREAM CONVERGING ELBOW



PIEZOMETER NUMBERS	DISTANCE FROM FACE
1, 5, E	1/64"
2, 6, F	1/16"
3, 7, G	5/32"
4, 8, H	31/64"
9, 10, I	0.977

TABLE I

PRESSURE DISTRIBUTION IN ENTRANCE

PIEZOMETER NUMBER	PRESSURE IN FEET OF WATER					
	WITHOUT TRASH-RACK	WITH RACK, PIER OPPOSITE E	WITH RACK, SPACE OPPOSITE E	RACK AT TOP POSITION	RACK AT DESIGNED POSITION	RACK AT BOTTOM POSITION
1	2.444	4.079	4.079			
2	0.610	1.988	1.920			
3	0.658	2.774	2.706			
4	1.789	2.051	2.039			
10	1.920	1.789	1.844			
D	15.136	15.398	15.791			
E						
F	1.794	1.026	0.856	0.864	0.986	1.067
G	2.837	2.233	2.098			
H	1.920	1.779	1.736			
I	1.265	1.224	1.196			
C	19.847	19.847	19.784			
7	7.941	7.548	7.679			

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FIGURE 13. EFFECT OF ADJACENT OUTLET AND TRASHRACK

would be short-sighted not to take advantage of the new apparatus and at least make check studies of gates for the outlets, even though the contract for the purchase of the gates for the lower tier had been awarded.

The 108-inch gates to be installed in the lower tier were of two types, namely, a ring-follower gate in the upstream position, and a paradox gate in the downstream position for normal service. The paradox gate was used to obtain ease of operation; but, because of its susceptibility to injury as a result of the complex mechanism, the ring-follower type was installed for emergency use.

Types of Slide Gates

The paradox gate consists essentially of a leaf with a wedge to force the leaf against the conduit to make the water seal. The operation is purely mechanical. Starting from the open position, the leaf is lowered on roller trains until it reaches a point directly in front of the conduit opening; at this point the leaf is held by stops, and a toggle allows the movable wedge to be further lowered forcing the leaf downstream, thus transferring the water load from the rollers to the gate seal.

The ring-follower gate is a slide gate with a following ring which comes into place when the gate is open, making the conduit an unbroken passage through the gate section. The housing is sufficiently deep to accommodate the ring when it is in the closed position. The water seal is a machined surface between the leaf and the face of the conduit, as in the case of an ordinary slide

gate. Since there is no wedge action, there is a considerable amount of leakage which is a disadvantage. Another disadvantage is the excessive initial cost of the hoisting mechanism necessary due to the high friction force.

During the course of the gate studies,[18] it was observed that the pressure in the chamber above the movable ring could be controlled by varying the clearance between the leaf and the housing. The clearance was adjusted to give a more economical hoisting mechanism, and a sealing ring was developed to form a positive water seal. The ring-follower gate, with the addition of the sealing ring, gave rise to the ring-seal gate which was used in the upper and intermediate tiers of Grand Coulee Dam.

The details of the gate studies are beyond the scope of this treatise. Since it is not contemplated to operate the outlets at partial gate openings, and as there is no contraction of the jet when the gate is in the open position, a length of pipe could have replaced the gate section without affecting the studies of the remainder of the conduits.

Pressure Determinations in Lower Outlets

The behavior of the lower tier of outlets insofar as river bed scour is concerned was found to be satisfactory on the 1:120 scale hydraulic model at Fort Collins. Furthermore, there appeared to be no reason to question the pressure distribution within the conduits as they discharged considerably below the surface of the tailwater, and hence, would be subjected to sufficient pressures in any flow condition.

Research on the solution of a pipe with a parabolic profile until complete the hydraulic model of a lower outlet in the Bureau laboratory, the model was modified for various testing. The distribution of pressures under various conditions of deflector and necessary alterations is contained in Figures 14 to 18 inclusive, and Figure 19 shows the calibration curve obtained from the model studies. The outlets in the lower tier were not lined with steel downstream from the control gates.

__Pressure Distribution in the Transitions and Upper Tiers__

The discussion to this point completes the consideration of the hydraulic properties of the lower tier of outlets, and the two upper tiers from the transition to the downstream side of the control gates. A presumption had been made by the laboratory staff that adverse flow conditions would occur in the outlets with parabolic profile in the upper tiers when no atmospheric outlets. The nature of these conditions was vague, apparently not subject to logical analysis methods, as it later developed, knowing the effect of the change in elevation due to the parabolic profile. Since the profile was without precedent, it was easy to err. An indication of the pressures in the upper tiers was obtained by allowing the model of the lower outlet to operate with deflector removed. Startling results occurred when sub-atmospheric pressures of 45 feet of water on the prototype were computed from the model measurements. Obviously, this low pressure is a fallacy, since absolute zero is approximately 34 feet of water below atmospheric pressure; but it did definitely indicate that pressures sufficient

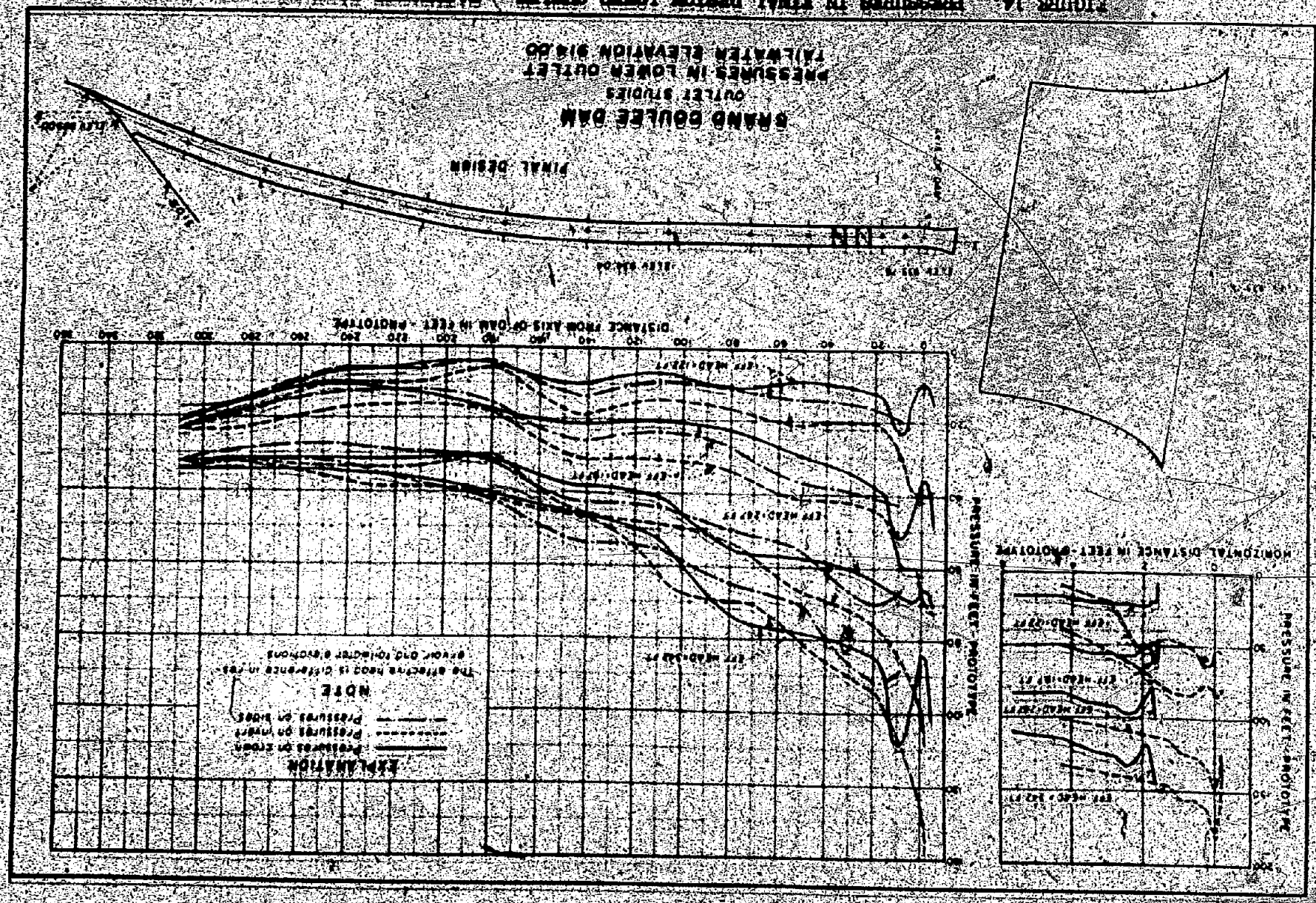


FIGURE 14. PRESSURES IN FINAL DESIGN LOWER OUTLET, MAXIMUM DISCHARGE 934.0
RAND DOUGLAS DAM
OUTLET STUDIES
PRESSURES IN LOWER OUTLET
TAILWATER ELEVATION 914.00
FINAL DESIGN
DISTANCE FROM AXIS OF DAM IN FEET - PROTOTYPE
EXPLANATION
Pressures on crown
Pressures on invert
Pressures on slope
NOTE
The effective head is difference in res-
ervoir and tailwater elevations
HORIZONTAL DISTANCE IN FEET - PROTOTYPE
PRESSURE IN FEET - PROTOTYPE

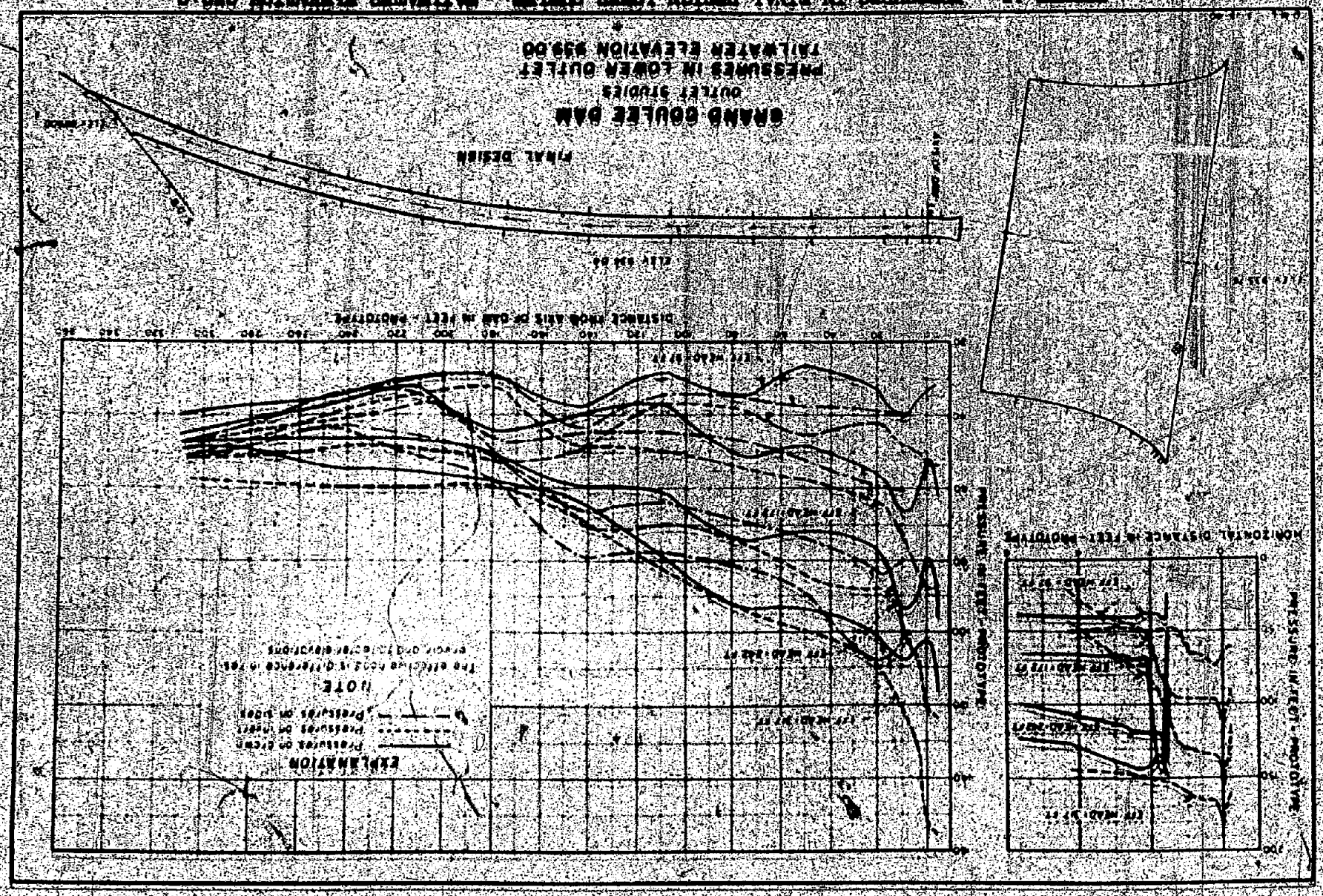


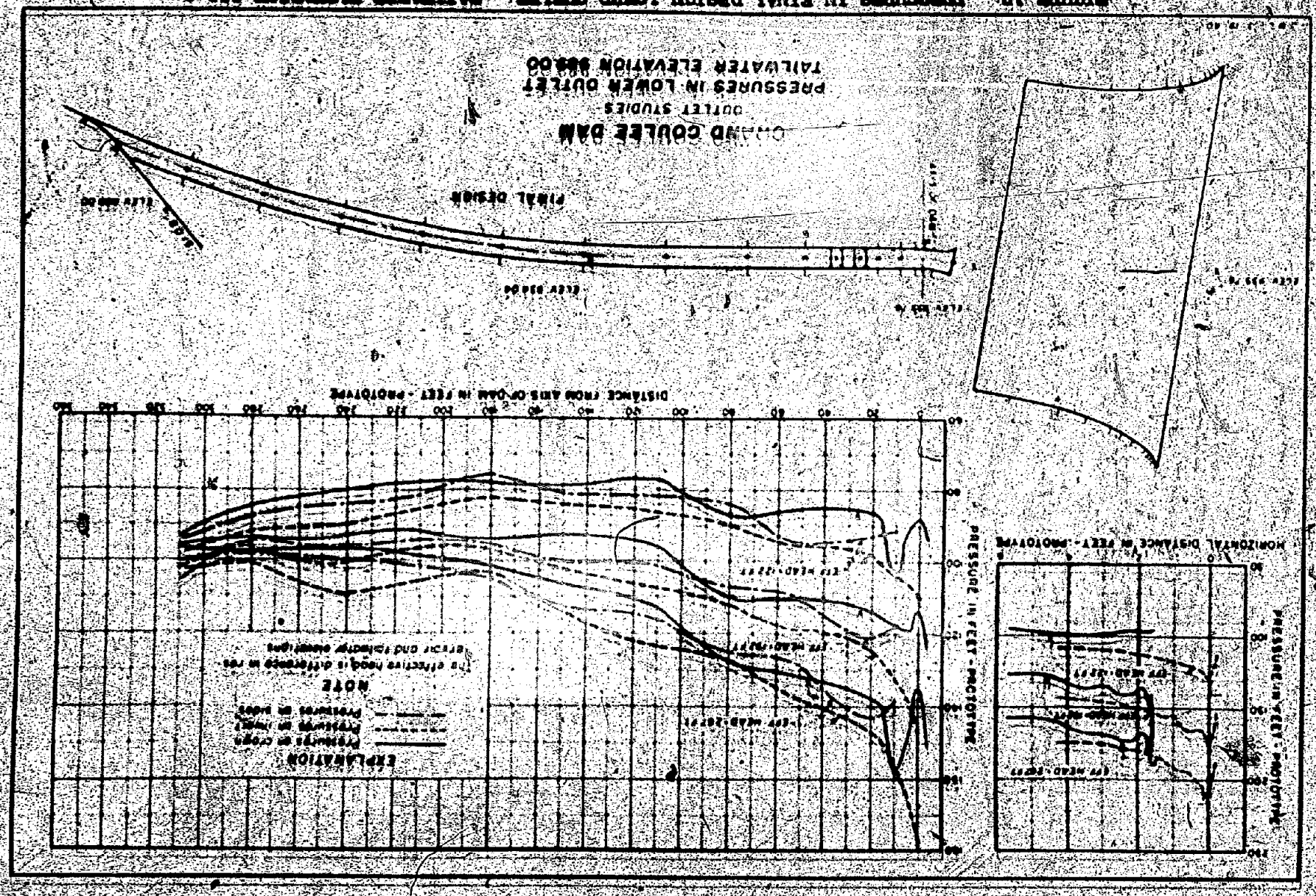
FIGURE 15. MEASURED IN FINAL DESIGN LOWER OUTLET. MAXIMUM ELEVATION 939.0

FIGURE 14. MEASURED IN FINAL DESIGN LOWER OUTLET. MAXIMUM ELEVATION 914.0

FIGURE 15. PRESSURES IN FINAL DESIGN LOWER OUTLET, MAXIMUM ELEVATION 949.0

FIGURE 15. PRESSURES IN FINAL DESIGN LOWER OUTLET, MAXIMUM ELEVATION 949.0

FIGURE 17: PRESSURES IN FINAL DESIGN LOWER OUTLET; TAILWATER ELEVATION 989.0



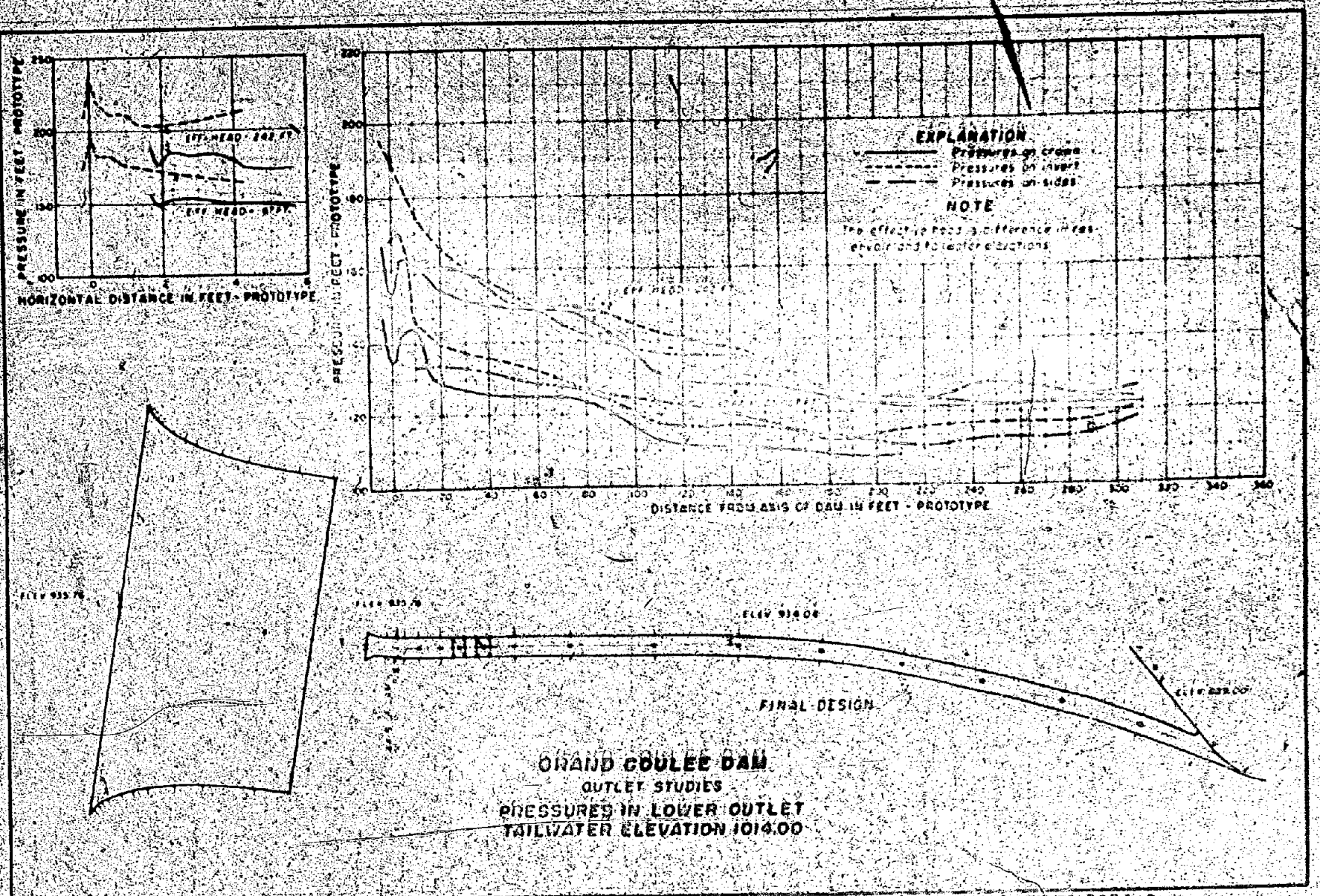


FIGURE 18. PRESSURES IN FINAL DESIGN LOWER OUTLET. TAILWATER ELEVATION 1014.0

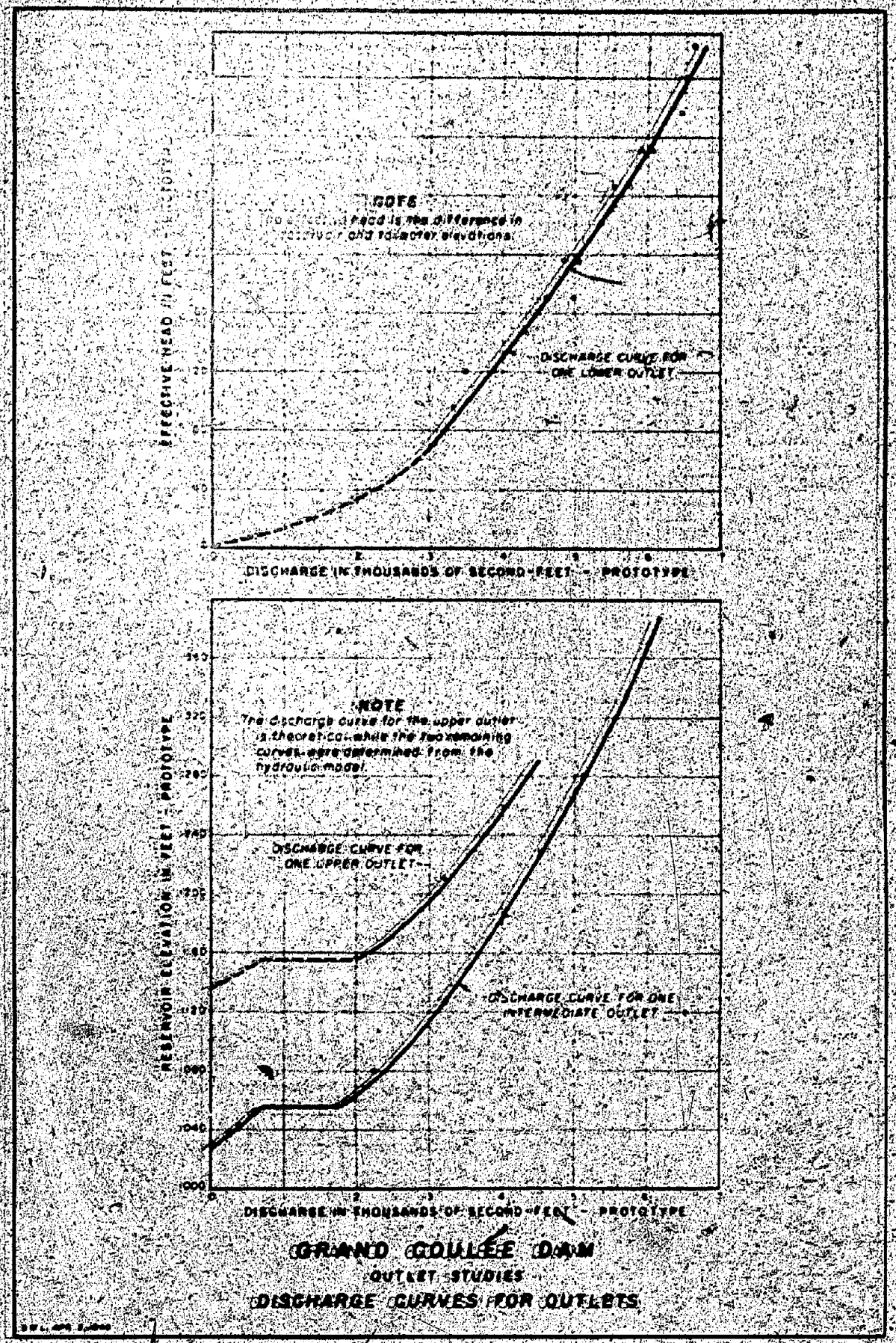


FIGURE 19. DISCHARGE CURVES

to cause severe cavitation would occur if this type of design was used without radical modifications. Excessively low pressures existed throughout the length of the conduit as shown on figure 20.

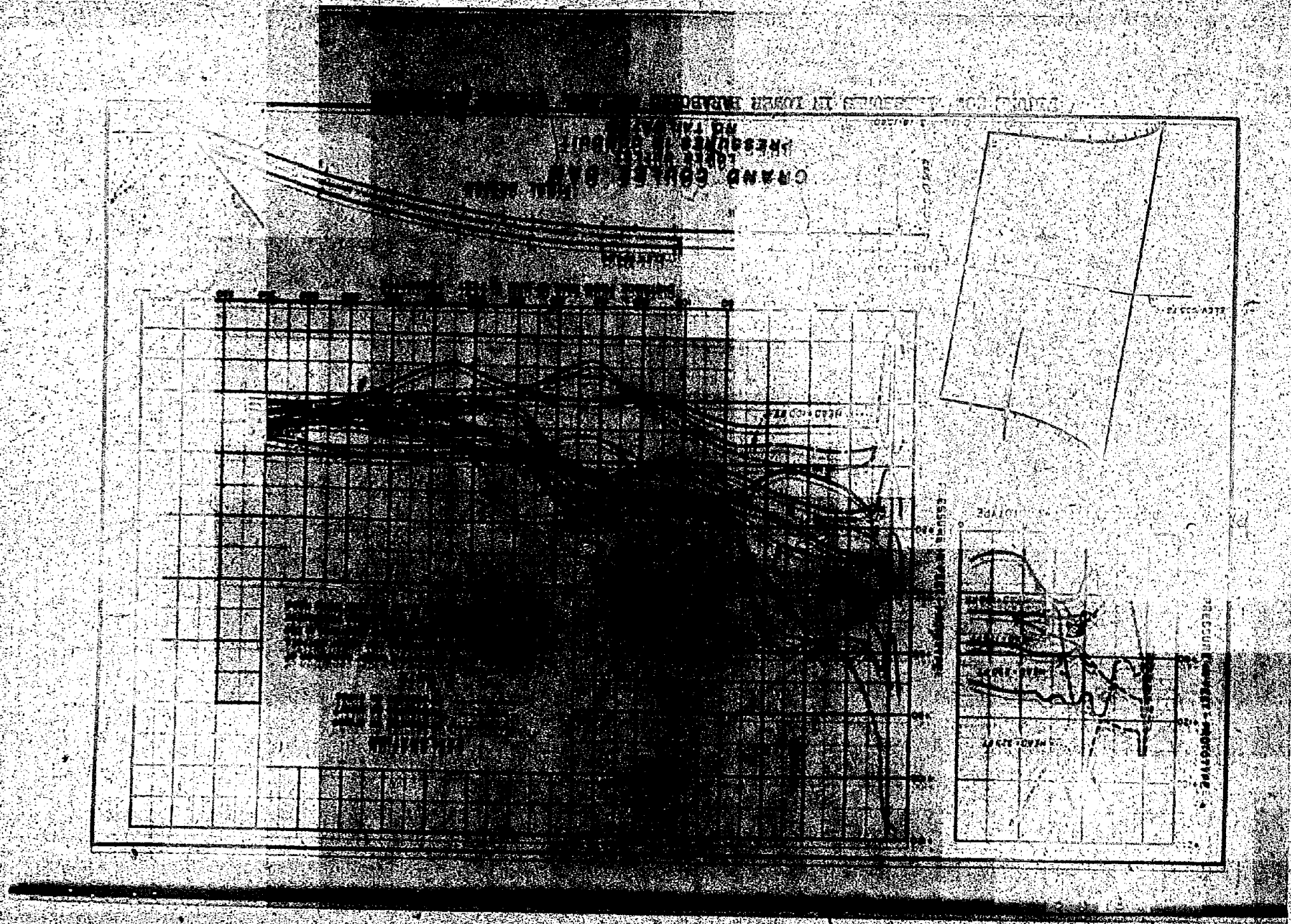
Apparently the original designer overlooked the fact that the frictional losses in the pipe were not sufficient to overcome the accelerating force which is relatively high because of the large drop in the conduit.

The results of the pressure studies gave convincing proof that the outlets with parabolic profile would not function properly in the two upper tiers without considerable revision of the downstream and to maintain positive pressures throughout the conduit during normal operating conditions.

As positive pressures within the outlet would force water through any existing cracks in the concrete and probably cause damage to the face of the dam, future studies were based on the decision that the intermediate and upper tiers would be lined with steel plates.

Another annoying condition to be considered in future studies was the formation of spray on the face of the dam. The canyon wall outlets at Boulder Dam demonstrated that the issuing jets released a spray which was carried up the canyon onto the electrical switching mechanism near the powerhouse.

Before proceeding with the new design, it was deemed advisable to show that a theoretical investigation, with proper assumptions, would give results similar to those obtained from the model



conditions the intermediate and upper tiers.

Theoretical Pressure Analysis of Outlets with Parallel Profile

Sufficient tailwater elevation to maintain positive pressures in the intermediate and upper tiers could never be obtained, although the intermediate outlets may occasionally be submerged, as the tailwater elevation varies over a wide range depending on the volume of water through the final control outlets and the powerhouse. In the case of submerged flow, air would not be admitted under any condition, to relieve these low pressures. In the case of free discharge, the negative pressure holds the jet to the walls of the conduit, preventing the intake of air until an unbalancing force is reached which is sufficient to cause the jet to break away from the sides.

The first condition will be analyzed by considering the intermediate tier when submerged, and the second condition with an analysis of the upper tier.

First condition:

Given:

Maximum reservoir surface, ... Elevation 1,990.00 feet

Exit of bypass, ... Elevation 973.97 feet

Tailwater assumed to seal exit, ... Elevation 997.00 feet

Equation of parabola, ... $y^2 = -81x$

Effective head ..... (1,990.00 - 997.00) = 993.00 feet

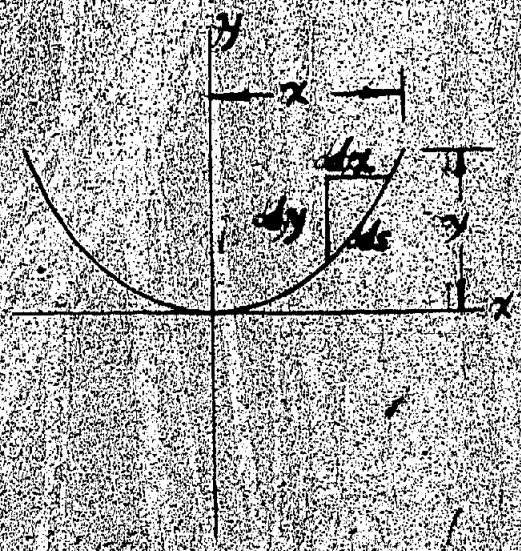
The assumed losses:

Loss in entrance and elbow, ... $0.10 \frac{v^2}{2g}$

Velocity head at entrance, ... $\frac{v^2}{2g}$

Frictional loss, ... $0.0075 \frac{L}{D} \frac{v^2}{2g}$

The length of a parabola is derived from the general equation, $y = ax^2$



$$\frac{dy}{dx} = 2ax$$

$$\frac{dy^2}{dx^2} = \frac{dy^2}{dx^2} + \frac{dx^2}{dx^2} = 4a^2x^2 + 1$$

$$ds = \sqrt{4a^2x^2 + 1}\ dx$$

$$s = \int_0^{x_1} \sqrt{4a^2x^2 + 1}\ dx$$

$$s = \frac{x_1}{2}\sqrt{4a^2x_1^2 + 1} + \frac{1}{4a}\log\left(2ax_1 + \sqrt{4a^2x_1^2 + 1}\right)$$

$$s = ax_1\sqrt{x_1^2 + \frac{1}{4a^2}} + \frac{1}{4a}\log\left\{2a\left(x_1 + \sqrt{x_1^2 + \frac{1}{4a^2}}\right)\right\}$$

where s = length of one-half of the parabola

The mean value of x_1 is [illegible] feet.

Hence $s = \frac{[illegible]}{2}\sqrt{4(0.[illegible])^2 + 1} + 204.00 \log(0.[illegible] + \sqrt{1.[illegible]})$

$= [illegible] (1.[illegible]) + 204.00 \log (1.[illegible])$

$= [illegible] + [illegible] (0.[illegible]) = [illegible]$ feet

The total length of the conduit is then:

$[illegible] + [illegible] = [illegible]$ feet

Writing Bernoulli's equation between entrance and exit and solving for velocity:

$$1.1\frac{v^2}{2g} + 0.0[illegible]\left(\frac{[illegible]}{[illegible]}\right)\frac{v^2}{2g} = 305.00 \text{ feet}$$

$$1.1\frac{v^2}{2g} + 0.217\frac{v^2}{2g} = 305.00 \text{ feet}$$

$$1.317\frac{v^2}{2g} = 305.00 \text{ feet}$$

$$\frac{v^2}{2g} = \frac{305.00}{1.317} = 230.07 \text{ feet}$$

Hence $v = 121.65$ feet per second

Head loss due to friction is:

$$0.217([illegible]) = [illegible] \text{ feet}$$

Drop of conduit from centerline at origin of parabola to tailwater elevation, ([illegible] - [illegible]) = [illegible] feet

Head loss from the origin of the parabola to the exit is:

$$0.0075 \frac{[illegible]}{[illegible]} \left(\frac{[illegible]}{[illegible]}\right)^2 = 40.[illegible] \text{ feet}$$

Hence the pressure gradient is, [illegible] - 40.[illegible] = 4.[illegible] feet below the centerline at the beginning of the trajectory.

Since the horizontal distance and the actual length of the conduit are nearly the same, the pressure gradient may be considered as a straight line (figure 21, sketch "B"). The maximum distance from the gradient to the centerline of the outlet is:

$$y = \frac{40.[illegible]\,x}{[illegible]} + \frac{4.[illegible]}{1} - \frac{x^2}{[illegible]}$$

$$\frac{dy}{dx} = \frac{[illegible]}{[illegible]} - \frac{x}{[illegible]}$$

For maximum, the derivative must be zero.

Therefore x = [illegible] feet

Substituting this value of x in the above equation:

$$y = \frac{40.[illegible]\,([illegible])}{[illegible]} + \frac{4.[illegible]}{1} - \frac{[illegible]^2}{[illegible]}$$

$$y = 12.[illegible] \text{ feet}$$

The pressure at centerline of the conduit at this point is, therefore, -12.[illegible] feet of water. The negative pressure at the top of the conduit at this section is -12.[illegible] plus the radius component.

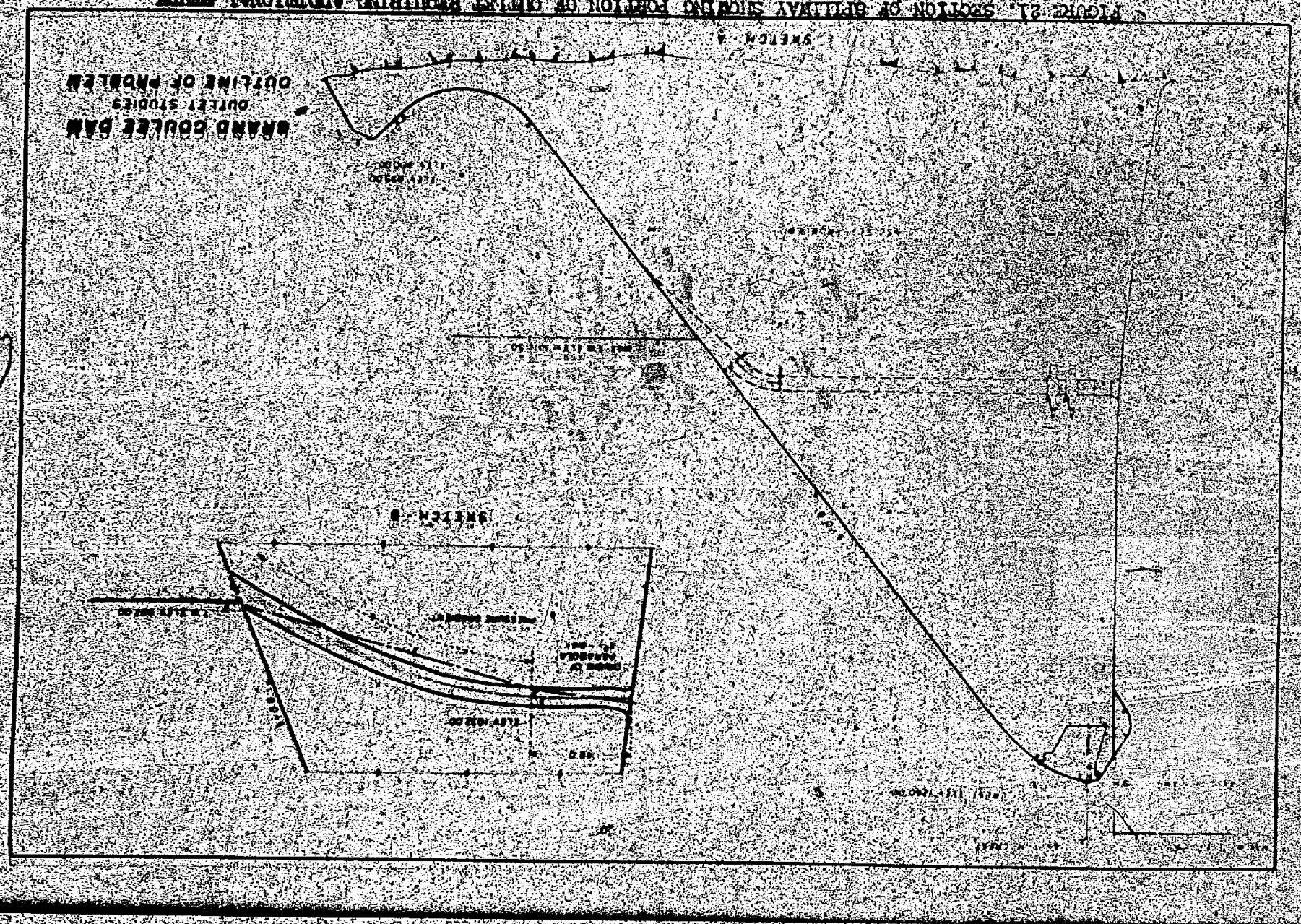


FIGURE 21. SECTION OF SPILLWAY SHOWING FOUNDATION OF OUTLET REQUIRING ADDITIONAL STUDY

The slope of the conduit is:

$$\frac{dy}{dx} = \frac{[illegible]}{[illegible]} = [illegible]$$

$$\arctan [illegible] = 11^\circ\ [illegible]'$$

The vertical component of the radius is:

$$\frac{[illegible]}{[illegible]} \cos 11^\circ\ 15' = 4.17 \text{ feet}$$

The resulting pressure on the crown will be:

$$-(4.17 + [illegible]) = -17.[illegible] \text{ feet}$$

Since authorities indicate that cavitation begins at a pressure of approximately one-half an atmosphere, the computation shows that pressures conducive to cavitation exist.

As the frictional resistance in the conduit creates a force tending to increase the pressures, the condition is aggravated by lowering the reservoir elevation to give a smaller friction loss. The clearness of this will be shown by a computation similar to the one above but with an effective head of 200 feet.

Again writing Bernoulli's equation between the entrance and exit and solving for velocity:

$$1.1 \frac{v^2}{2g} + 0.0075 \left(\frac{[illegible]}{[illegible]}\right) \frac{v^2}{2g} = 200.00$$

$$1.1 \frac{v^2}{2g} + 0.317 \frac{v^2}{2g} = 200.00$$

$$1.317 \frac{v^2}{2g} = 200.00$$

$$\frac{v^2}{2g} = \frac{200.00}{1.317} = 151.86$$

FIGURE 21. SECTION OF SPILLWAY SHOWING PORTION OF OUTLET REQUIRING ADDITIONAL STUDY

Head [redacted] feet per second

Head loss due to friction is:

[redacted] = [redacted] feet

Head loss from origin to exit is:

[redacted] = [redacted] feet

As the drop from the centerline at origin of parabola to tailwater surface is 45.00 feet, the pressure gradient is:

45.00 - [redacted] = [redacted] feet below the centerline at the beginning of the trajectory.

The minimum distance from the gradient to the centerline of the outlet is:

[redacted]

[redacted] = 0

Therefore x = [redacted] feet

Substituting this value of x in the above equation:

[redacted]

y = [redacted] feet

The pressure at the centerline of the conduit at this point is -41.96 feet of water. The negative pressure at the top of the conduit at this section is -41.96 plus the radius component. As previously found the radius component is 4.17 feet.

The resulting pressure on the crown is:

$$+(4.17 + 21.[illegible]) = +[illegible] \text{ feet}$$

By decreasing the effective head 100 feet, the maximum subatmospheric pressure was increased 9.12 feet. Hence, it is apparent that decreasing the head on the outlet drops the hydraulic gradient.

Second condition:

An investigation at the upper tier where no submergence is possible will show a different result:

Given:

Reservoir surface	Elevation 1,[illegible].0 feet
Exit invert	Elevation 1,[illegible] feet
Head on outlet (1,[illegible].0 - 1,103.8),	[illegible] feet
Length of conduit	170.0 feet

Assuming the same losses as before, Bernoulli's equation gives:

$$1.1\frac{v^2}{2g} + 0.0075\left(\frac{170}{8.5}\right)\frac{v^2}{2g} = 186.2 \text{ feet}$$

$$1.1\frac{v^2}{2g} + 0.15\frac{v^2}{2g} = 186.2 \text{ feet}$$

$$\frac{v^2}{2g} = \frac{186.2}{1.25} = 148.96 \text{ feet}$$

$$v = 97.88 \text{ feet per second}$$

$$\text{Head loss due to friction} = 0.0075\left(\frac{170}{8.5}\right)\frac{(97.88)^2}{64.32} = 22.34 \text{ feet}$$

The drop in the passage is 1136.67 - 1112.17 = 24.50 feet

Since the drop in the passage exceeds the head loss due to friction by 2.16 feet, there remains an unbalanced force. By the same reasoning used in the first condition, this force will become greater as the reservoir elevation is decreased. If the force becomes great enough to cause the water to break away from the sides of the conduit, atmospheric pressure will be admitted. The jet will carry the air and until a pressure sufficiently low occurs to again allow air to be drawn into the conduit, thus resulting in an unstable condition of flow.

IV— OUTLETS HAVING HORIZONTAL PROFILE WITH ELBOW AND CONTRACTED EXPANSION

Summary of Problem

The problem now resolved itself into the evolution of a conduit design between the downstream end of the control gates and the face of the dam which would:

1. Prevent the formation of sub-atmospheric pressures in the outlet,
2. Minimize the formation of spray,
3. Minimize the river bed erosion.

The ideal solution was to bring the outlets through the dam on a horizontal profile, and curve the conduits to allow the issuing jets to flow parallel to the face of the dam. The outlet streams following down the spillway face would be less conducive to the formation of spray than one falling freely through space, and the river bed erosion would be minimized because of the energy dissipation in the spillway bucket. Since this would have necessitated an elbow protruding onto the face of the spillway causing severe disturbance when the spillway was operating, the practical solution required a compromise between the two flow conditions.

Limits of Design

Simplicity of design and prevention of the undesirable conditions outlined above indicated the desirability of placing the conduit on a horizontal profile with a short-radius elbow at the downstream end to cause the high-velocity jet to discharge nearly parallel to the face of the spillway. The proximity of the elbow to the spillway face was

determined by the thickness of material necessary for structural purposes.

A section of the spillway including the uncompleted portion of the problem is shown on figure 21. If the elbow were removed allowing the jet to discharge into the atmosphere, the first condition would be satisfied. Hence, the problem has now been reduced to the designing of an elbow with a constriction to maintain positive pressures in the conduit, and an exit channel in the face of the spillway which would satisfy the remaining two conditions.

The length of the open channel portion of the intermediate outlet was limited, since the prototype structure had been completed to elevation 975 when the difficulties were discovered in the permissible design. The issuing jet could not be placed on a trajectory in this short distance; therefore, it was desirable to make the opening in the spillway face as short as possible to minimize interference with the spillway jet.

So far as is known, the only study previously made on an outlet which bears any resemblance to these studies was that in the laboratory of the Carnegie Institute of Technology for the United States Engineer Office[13] in connection with the design of Tygart and Bluestone Dams. The structure resembled a tetrahedron which served to choke the opening and create a fan-shaped jet to dissipate the energy. The use of such a structure would be contrary to the second condition.

Conducting Ribs

From the preceding computations it can be seen that the percent of

area reduction required to maintain positive pressures in the conduit increased as the head causing the flow diminished; but the maximum discharge was also decreased. Accordingly, it was necessary to establish a minimum head under which the outlets would normally operate. After consideration of the flow downside in the river downstream from the dam, this head was fixed at 40 feet, with the understanding that the restriction of head would not apply during the construction period.

Since a sudden change in the direction of a conduit may cause an unfavorable pressure distribution,[24] the reduction in area was obtained, for the first trial, by converging the crown of the elbow onto the invert. By making the necessary assumptions in Bernoulli's equation, the area of the outlet end of the elbow was estimated to be 28.48 square feet. As the area at the upstream end of the elbow is 34.25 square feet, this corresponded to a reduction in area of 16.8 percent. A minimum distance of three feet was established between the elbow and the face of the dam to allow for the placement of reinforcing steel. The invert of the open-channel portion of the outlet was made circular in cross section, and tangent to the invert of the elbow.

The pressures in the elbow were above atmospheric (figure 22, design 1), but the discharge was materially decreased. The fins of water which formed on both sides of the jet in the upper end of the open channel were objectionable because of the spray released, and because of the unpleasant appearance which would result when all forty outlets were discharging.

In an effort to prevent the fins from forming, the open channel was made as before at the upstream end, and warped into a flat

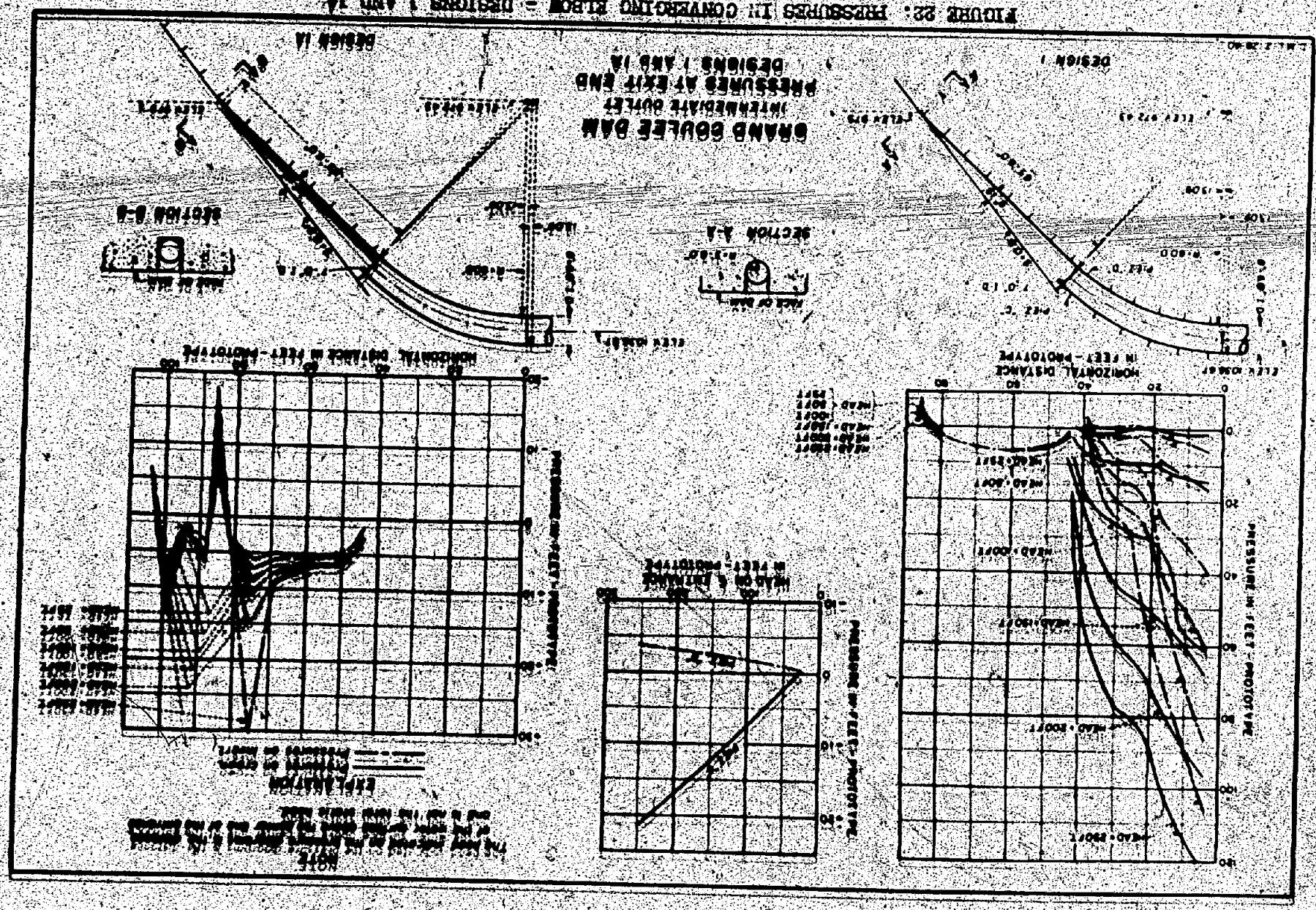


FIGURE 22: PRESSURES IN CONVERGING ELBOW - DESIGNS 1 AND 1A

section at the intersection with the face of the dam (figure 22, design 14). The fins persisted in forming and the invert pressures at the lower end of the channel was extremely low.

Excessive cost of fabrication and installation was foreseen in the construction of an elbow with a constantly reducing cross section. A constant-section elbow with a reducing cone on the end would eliminate these difficulties. Assuming the hydraulics to be the same for the two cases, the latter was preferable.

Constant Section Elbow with Enclosed Converging Cones

The constantly reducing elbow was abandoned in favor of a constant-section elbow with a reducing cone on the end, the reduction in area being 25.24 percent. As it seemed logical to keep the invert of the outlet on a smooth curvature, that is, no sudden changes in direction, the crown of the cone was converged onto the invert (figure 23, design 2). On the supposition that the fins were caused by pressure exerted by the channel sides on the jet, the open channel was diverged in section to relieve this pressure. The invert of the apron was circular in section at the upper end, terminating in a flat section at the lower end.

The resulting measures (figure 23) in the elbow and cone were of sufficient magnitude to allow the exit area to be increased, however, the objectionable fins occurred on both sides.

The exit channel was then made with a constant cross section including a circular invert for a distance of approximately 22 feet,

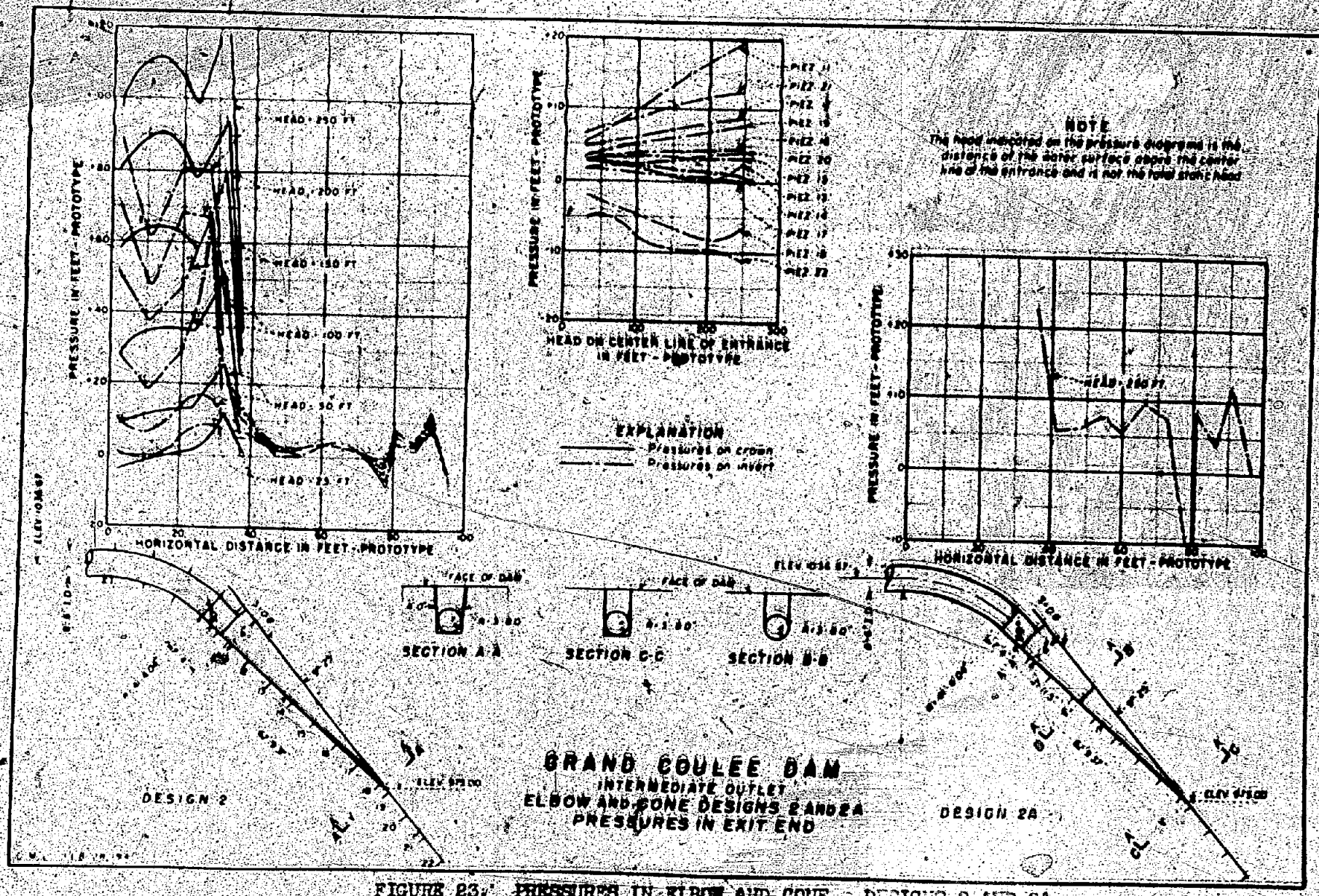


FIGURE 23. PRESSURES IN ELBOW AND CONE - DESIGNS 2 AND 2A

terminating into a flat section at the lower end (figure 23, design 2). When the conduit was discharging under a head of 200 feet, the flow extended a distance of 20 feet from the face of the dam.

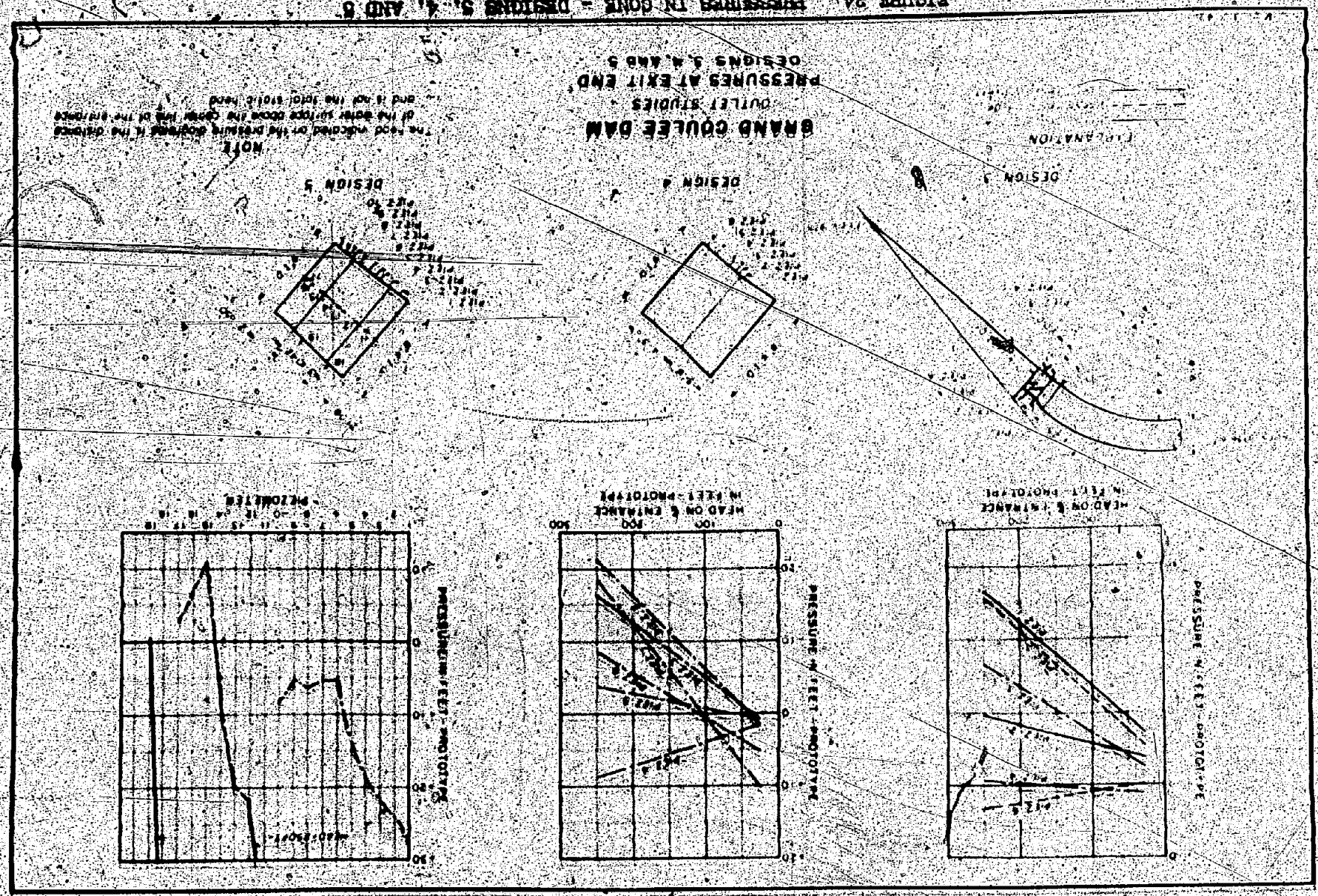
The results of the latter design indicated that the flow was caused by excessive pressure on the invert rather than on the sides, as originally believed. The pressure was due to the mass of the cone converging into the invert. To relieve the pressure at the upper end of the cone, the converging cone was made symmetrical about the centerline, thus forming a slight, but definite, break at the intersection of the cone and the open channel (figure 24, design 3).

Since the flow did not form on the latter design, it was reasonable to conclude that the bottom pressures were the cause of the water being forced out to a region of low pressure on the sides, resulting in the formation of flow. Sub-atmospheric pressures were expected to occur in the region immediately below the break, but due to the urgent need of obtaining a satisfactory cone for use by the mechanical section in completing the details of the prototype design, these pressures were not studied until later.

Design 3 consists of a right circular cone with a short length of cylinder added to the downstream end. As may be seen on the pressure diagram, the jet continued to contract after passing the cone, causing excessive sub-atmospheric pressures in the constant section downstream.

In designs 4 and 5 other lengths of convergence were tried;

FIGURE 24. PRESSURES IN DUCTS - DESIGNS 3, 4, AND 5



the pressure diagrams show that the contraction of the jet decreases as the length of the converging section increases. In the final design, the cone was lengthened and the downstream constant section eliminated, which reduced the number of welded joints. This latter factor is of importance as each joint is a source of trouble.

Final Elbow and Cone Design

This final design of cone (figure 20) was installed on the hydraulic model and a complete study made of the pressures under various operating heads. The pressure diagram indicates super-atmospheric pressures for all heads of 40 feet or above on the bellmouthed entrance. The sub-atmospheric pressures occurring when the outlet is discharging under a head less than 40 feet should do no damage in the short period of operation necessary during construction. Figure 19 shows the rating curve for one intermediate outlet.

Pressure curves for the portion of the outlet upstream from the elbow are shown on figure 26; the measurements were taken with the air vents at the gates closed. Under this condition the minimum head on the centerline of the bellmouthed entrance required to cause the conduit to flow full at all points was 21.42 feet, however, when the air vents were open the conduit flowed only partially full under a head of 28 feet. With the latter condition the intake of air was intermittent, resulting in an unsteady flow condition.

The final conduit design which has been described was adopted for the intermediate tier of outlets and, except for a last minute change which will be described later, was adopted for the upper tier at Grand Coulee Dam and all three tiers at Shasta Dam.

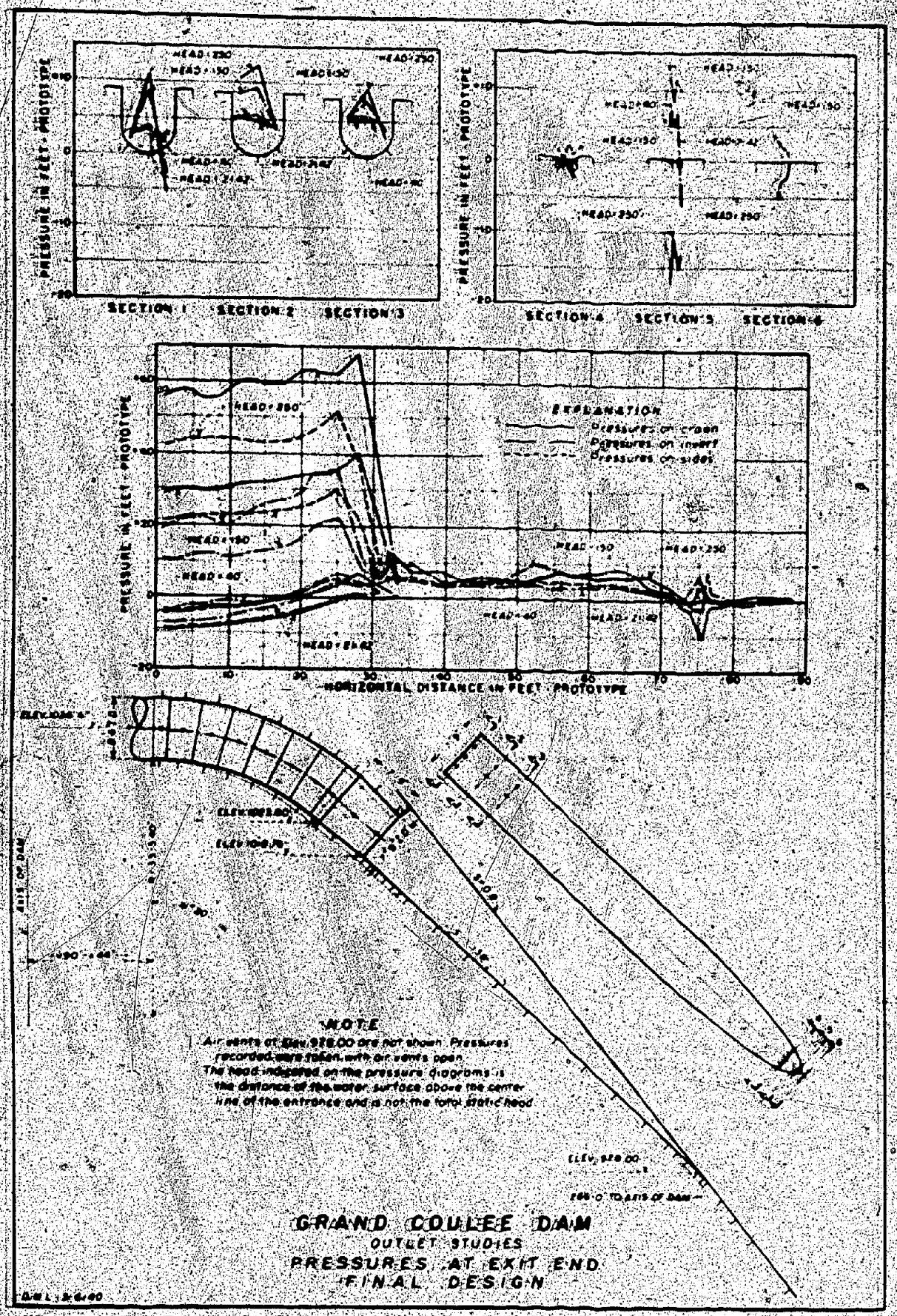


FIGURE 25. ELBOW, CONE, AND APRON PRESSURES - FINAL INTERMEDIATE OUTLET

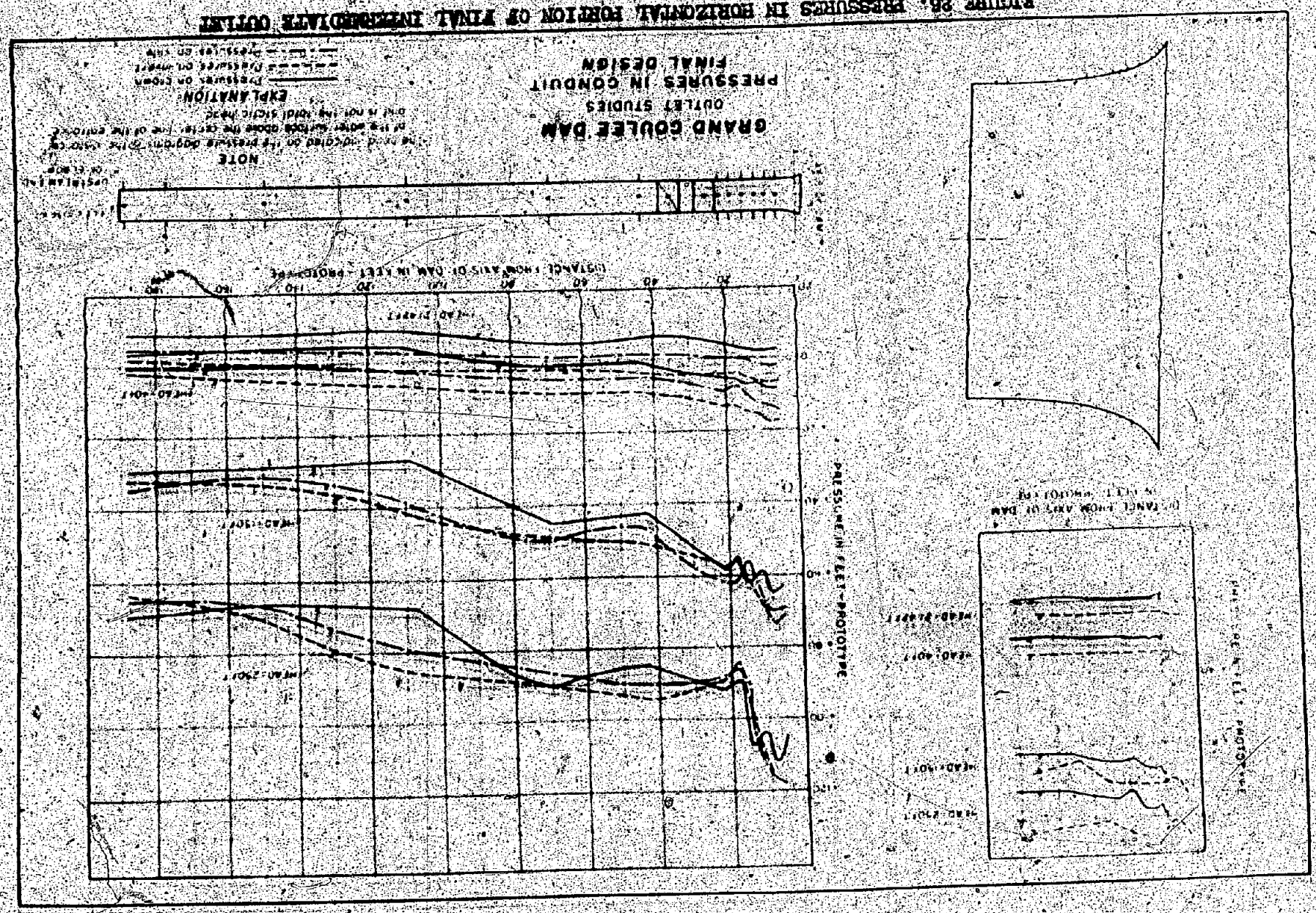


FIGURE 26. PRESSURES IN HORIZONTAL PORTION OF FINAL INTERMEDIATE OUTLET

Open-Channel Portion of Outlets

The open-channel, or apron, portion of the outlets presented a problem of greater magnitude than anticipated. As the solution of the elbow and exit channel was unfolded, the realization of the importance of the shape of the open channel became increasingly apparent. In the case of ordinary open channel flow, minor surface irregularities are of relatively little importance, but the present problem required a discharge surface which would reasonably fit a jet of water 8 feet, 6 inches in diameter and flowing at a maximum velocity of approximately 85 feet per second. Any non-conformities tending to change the direction of this posterous jet would result in unfavorable flow conditions.

An apron with an invert whose radius at the upper end equalled the radius of the lower end of the cone and with a warp to a flat section at the lower end was installed on the conduit of the final design. A sharp break occurred at the intersection of the cone with the apron, and also at the junction of the apron with the face of the dam (Figure 27, design A). The pressures immediately downstream from the cone were slightly super-atmospheric which was contrary to the expectation that severe sub-atmospheric pressures would occur in this region. However, excessively low pressures occurred on the apron near the intersection with the face of the dam, which was a complete surprise.

An apron with an invert radius throughout its length equal to the downstream radius of the cone was next tried (design B). The

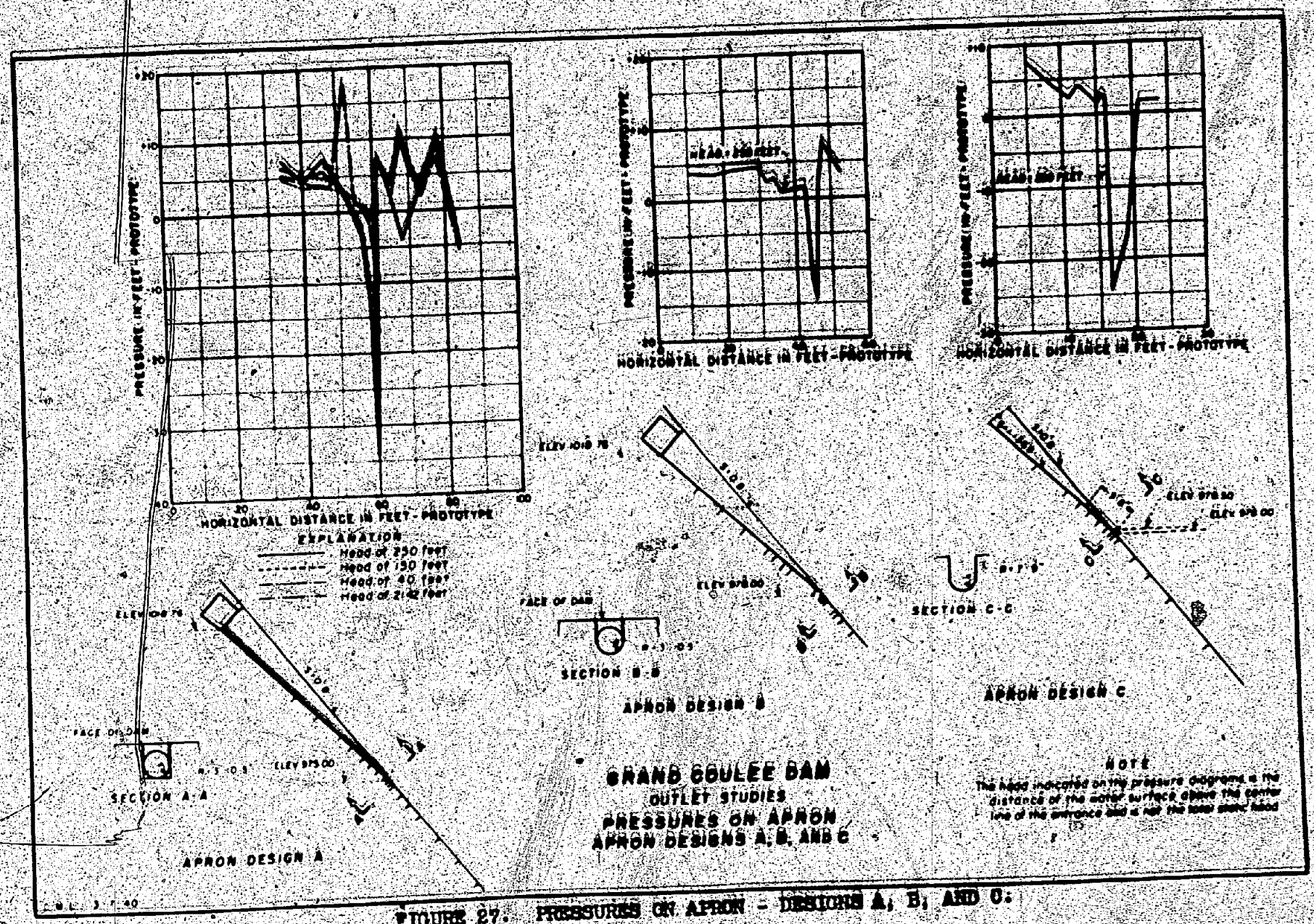


FIGURE 27. PRESSURES ON APRON - DESIGNS A, B, AND C.

pressure curves show that sub-atmospheric pressures still exist at the lower end. Logical reasoning dictated that if the lower end was turned upward into the jet (design C), positive pressures would result on the apron, and at the same time allow air to enter beneath the jet to aerate that portion of the face of the dam. The pressure curves of this trial showed that sub-atmospheric pressures still existed in this region.

The apron was then connected to the face of the dam with a short curve (figure 28, design D). Since the pressures on the curved portion were below atmospheric pressure, it follows that the high velocity jet will not follow such a steep trajectory.

The reason for the negative pressures occurring in the area immediately downstream from the intersection of the apron with the face of the dam is quite apparent. The high-velocity jet naturally would not follow the slight angle between the invert of the apron and the face of the spillway. On the other hand, the jet clung so closely to the face of the spillway as to not permit air flow and relieve the sub-atmospheric pressures at this point. Figure 29, A, shows the jet on the face of the model in this region and figure 29, B, indicates the same action on the prototype.

The reason for the low pressure on the apron adjacent to its intersection with the face of the dam was not as apparent. In figure 27, design A, the pressures at the upstream end of the apron are slightly super-atmospheric; further downstream they are increased in intensity; and finally at the end the pressures are

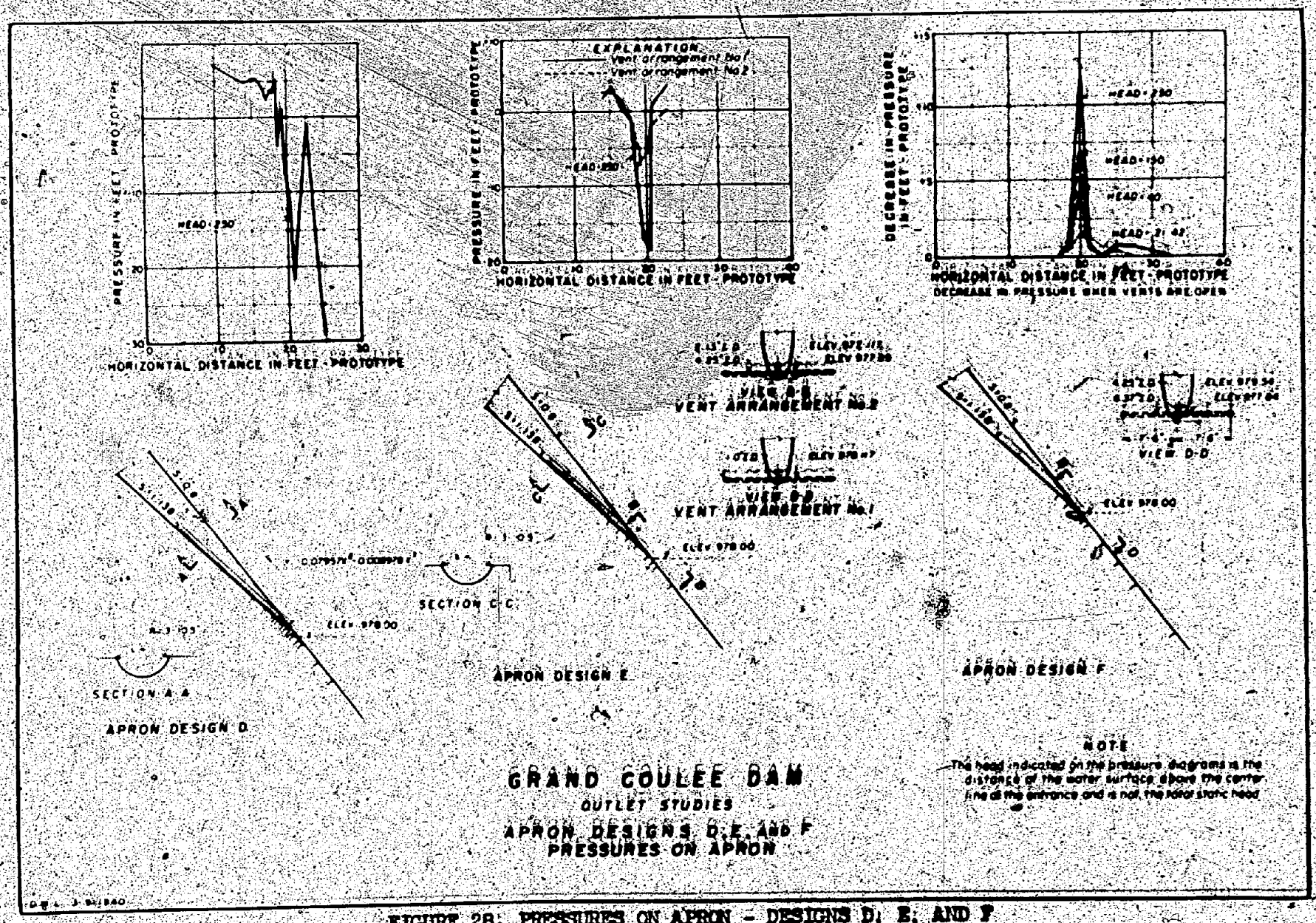


FIGURE 28: PRESSURES ON APRON - DESIGNS D, E, AND F

severely sub-atmospheric. The explanation of this condition is that the slight angle between the invert of the cone and apron causes the jet to impinge slightly at the first region of high pressure and ricochet causing the low pressure on the channel near its intersection with the spillway face.

Final Design Area

Visually, there was no evidence of this ricocheting action, but a line of piezometers along the center of the invert indicated the presence of the low pressure. Additional piezometers were installed as the tests progressed until the extent of the low pressure region was defined. It then became a tedious proposition of finding a means of either preventing the formation of the negative pressures, or of relieving them, a task which in retrospect looks comparatively simple.

The elimination of this negative pressure area proved to be quite a puzzle and required considerable time, mainly because of the dearth of ideas on the possible solution, and when the solution did develop it was completely by accident. The pressures in the elbow, cone and upper portion of the channel were good, the jet of water was stable and undisturbed, but this low pressure area was the "fly in the ointment". Furthermore, the tests on other shapes of outlets had shown that the optimum design had been reached so far as structural design and visual observations were concerned.

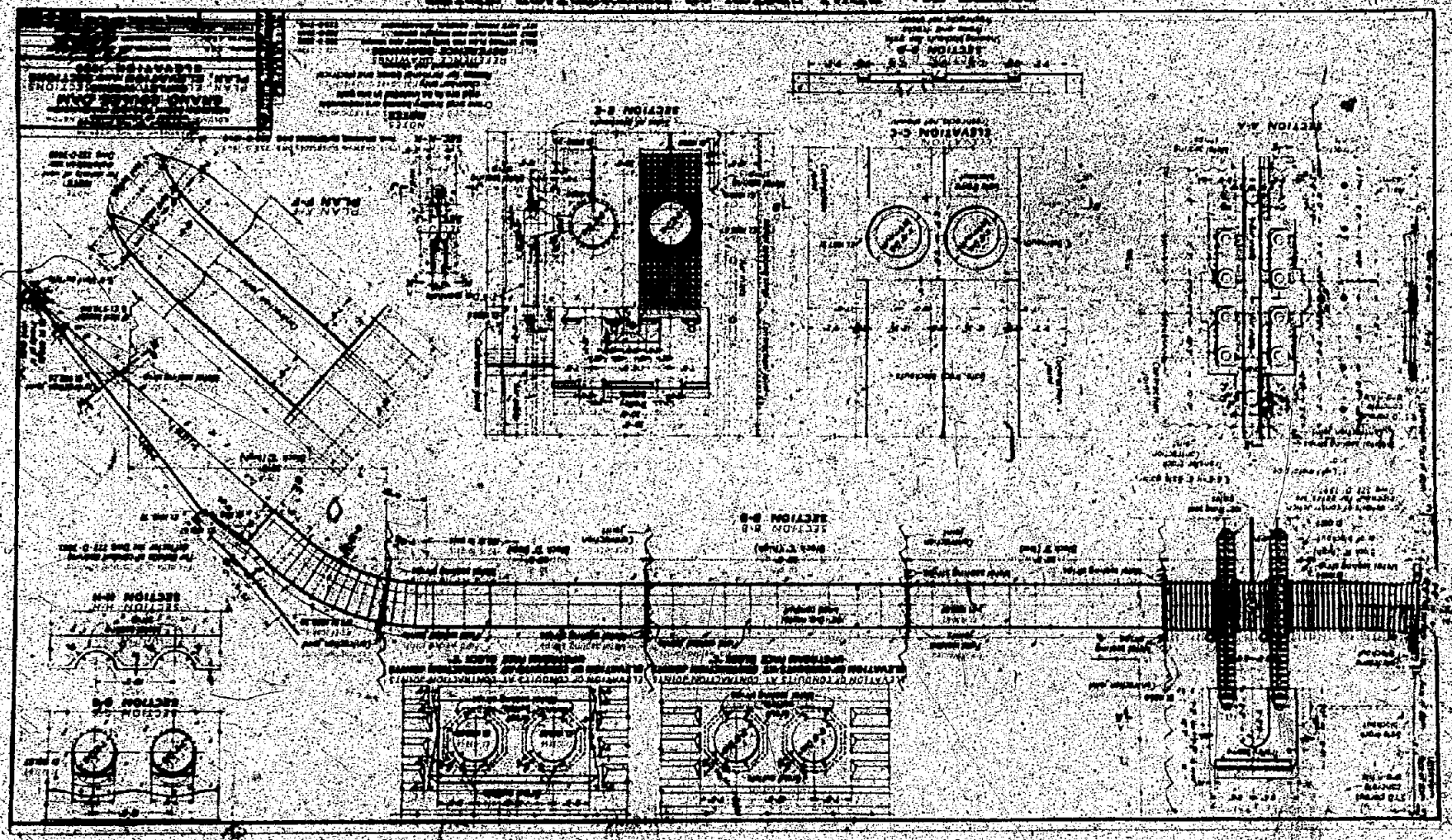
Subsequent to several weeks of study, the glass piezometer tubes were replaced by U-tubes to eliminate the necessity of establishment of a datum plane after each change of design on the model. During

the process of changing the rubber connecting tubes to the manometers, it was accidentally observed that the pressures immediately downstream from any disconnected tube were increased. An investigation revealed that air was being drawn through the disconnected tubes, creating the region of low pressure. Consequently, it was decided to install air vents for the sole purpose of relieving the negative pressures. Although this solution was accidentally discovered, the detection of the presence of such a phenomenon would have been impossible without the model studies.

The air supply was taken from the face of the dam at some distance on either side of the tunnel to prevent the outlet jets from covering the openings (Figure 30, plan F-F). On the hydraulic model, the size of the air conduits was determined by experimentation (figure 26, designs 3 and 7). Very little is known concerning the quantity of air required in the prototype to relieve sub-atmospheric pressures compared to that of the model; hence, by direct ratio of the diameters, the prototype header lines were made with a diameter of 6 inches while the feeder lines were of 4-inch diameter pipes. Figure 26, design 7, shows the decrease in sub-atmospheric pressure when the jet air-supply vents are open, while figure 25 shows the pressure curves for the final design apron with air vents open.

The jet air-supply piping is arranged to be self-draining to prevent water from accumulating in the lines when the outlets are not in operation (figure 30). It is apparent that no air may be taken through the air lines while the spillway is operating; however, under this condition the vertical component of the mass of the spillway

FIGURE 30: FINAL DESIGN OF INTERMEDIATE OUTLET



jet will force the outlet jet downward, thus preventing sub-
atmospheric pressures.

V — EFFECT OF SPILLWAY ON OUTLETS

Simultaneous Operation of Outlets and Spillway

When the outlets and spillway are operated simultaneously, one of two conditions is possible. If the spillway water is flowing faster than the outlet jet, an ejector action will be created resulting in an increased discharge through the conduits. Conversely, if the spillway flow is slower than the outlet jet, the discharge of the latter will be decreased. In the case of the former condition, the conduit would undergo a pressure reduction due to the higher velocity. A computation will show which condition exists in the problem at hand.

Theoretical Determination of Velocity of Spillway Flow

The relation between the velocity and the length of a steep slope, considering the air resistance, is given by:[15]

$$\frac{dl}{dv} = \frac{\frac{\alpha v}{g} - \frac{Q}{v^2}\cos\theta}{\sin\theta - \frac{\frac{\alpha v^2}{g}}{2.2088\, R_{m=0}^{2/3}} - \frac{0.0000014\, v^2}{R_{m=0}^{1.46}}}$$

Where, l = length = 990.15 feet

Q = discharge per foot of crest = 873.4 second-feet.

g = acceleration of gravity = 32.16 feet per second per second

n = coefficient of roughness, assumed = 0.011

α = coefficient which multiplied by mean velocity head will give distance of energy head above water surface = 1.07

$\theta = 11° - 30'$

$\sin\theta = 0.[illegible]$

$\cos\theta = 0.[illegible]$

R = hydraulic radius = depth of flow with no air in water

In making this computation the face of the dam was assumed to be a straight line below elevation 1260.00 and no loss was assumed between the reservoir and this elevation.

Writing Bernoulli's equation between elevations 1260.00 and 1200.00, and solving for velocity:

$$\frac{v^2}{2g} = [illegible]\cos 11° - 30' + \frac{v^2}{2g}$$

$$\frac{v^2}{2g} = [illegible](0.[illegible]) + \frac{v^2}{2g}$$

$$[illegible]v = 57,[illegible] + v^2$$

$v = [illegible]$ feet per second

Substituting the known values in the original equation and evaluating gives:

$$[illegible]v = \frac{[illegible]}{v^2}$$

$$\frac{1}{n} = \frac{[illegible]}{0.[illegible] - 0.[illegible] v^{1.5/3} - 0.[illegible] v^{3.46}}$$

Figure 31 shows the results of the computations. By interpolation the velocity at the intersection of the spillway jet with that of the outlet is found to be 135 feet per second.

Theoretical Determination of Velocity of Intermediate Outlet Jet

The discharge velocity may be expressed by the relation:

$$H = 0.10\frac{v^2}{2g} + f\frac{l}{d}\frac{v^2}{2g} + \frac{d^4}{d_1^4}\frac{v^2}{2g}$$

$$H = \frac{v^2}{2g}\left[0.10 + f\frac{l}{d} + \frac{d^4}{d_1^4}\right]$$

Where, H = total head on exit end = [illegible] feet at maximum discharge

$0.10\frac{v^2}{2g}$ = loss due to entrance and bends

f = friction factor based on Reynolds' number = 0.016

l = length of conduit = 220.75 feet

d = diameter of conduit = 8.5 feet

d_1 = diameter of conduit at exit end = 7.75 feet

v = velocity

g = acceleration due to gravity = [illegible] feet per second per second.

Substituting the values and solving for the velocity:

$$[illegible] = \frac{v^2}{[illegible]}\left[0.10 + 0.016\frac{220.75}{8.5} + \frac{8.5^4}{7.75^4}\right]$$

v^2 = [illegible]

v = [illegible] feet per second

Analysis of Velocity Determinations

The computations indicate that at maximum reservoir elevation, the spillway flow will be approximately 29 feet per second faster than the intermediate outlet jet when the two intersect. With a

lower reservoir elevation, the percent of decrease of the head on the outlet is small, while the percent decrease of the head on the spout will be appreciable. Since a decrease of head corresponding to a lower value of Reynolds' number gives a higher friction factor, a condition will exist when the spillway flow will be slower than the outlet jet. The latter condition may cause a decrease of velocity in the outlet with the corresponding rise of pressure within the conduit, but this would be on the side of safety. The other condition may result in an increase in velocity through the conduit accompanied by a reduction in pressure. The cycle of operation allows either condition to occur.

The limitations of the model prevented the study of the pressure distribution within the conduit under the above conditions, and the construction of a model of sufficient size to give reliable results would not be justified by the returns.

Operation of Spillway with Outlets Closed

When the outlets were closed and the spillway jet flowed over the openings in the face of the dam, a partial vacuum occurred in the conduits. Also, while the ring-seal gates were being closed or opened an abundance of air would be required immediately downstream from the gates to prevent intermittent excessively low pressures from forming and causing vibration. To relieve these conditions, two air ducts were installed in each outlet at Grand Coulee Dam, one on the downstream side of each gate.

Size of Air Ports Required

Studies on a preliminary design of the upper outlets were made on a model of 1:60 scale. The amount of air required to relieve the low pressure in the conduit when the spillway was operating was studied in some detail, and the results were used in determining the size of air piping for later designs. The deflectors of the final design, which will be described subsequently, had not been studied at this time, hence, they were not included in the model. Figure 28 shows the relation between the diameter of orifice admitting the air and the corresponding pressure in the outlet for discharges of 700,000 and 1,400,000 second-feet over the spillway.

In studying the air demand, it was assumed that the laws of similitude pertaining to water also applied to air; this assumption may be questionable, but time did not permit exploring this field to gain the necessary experience for transference of results by any other method.

Theoretical Determination of Size of Air Duct

The discharge per foot of spillway crest is $\frac{1,400,000}{1535} = 911.62$ second-feet. The width of the open channel portion of the outlet is 7.75 feet.

Hence the amount of water passing over each outlet is:

$$7.75\ (911.62) = 4,730 \text{ second-feet}$$

Assuming that the lower one-quarter of the jet will pick up a

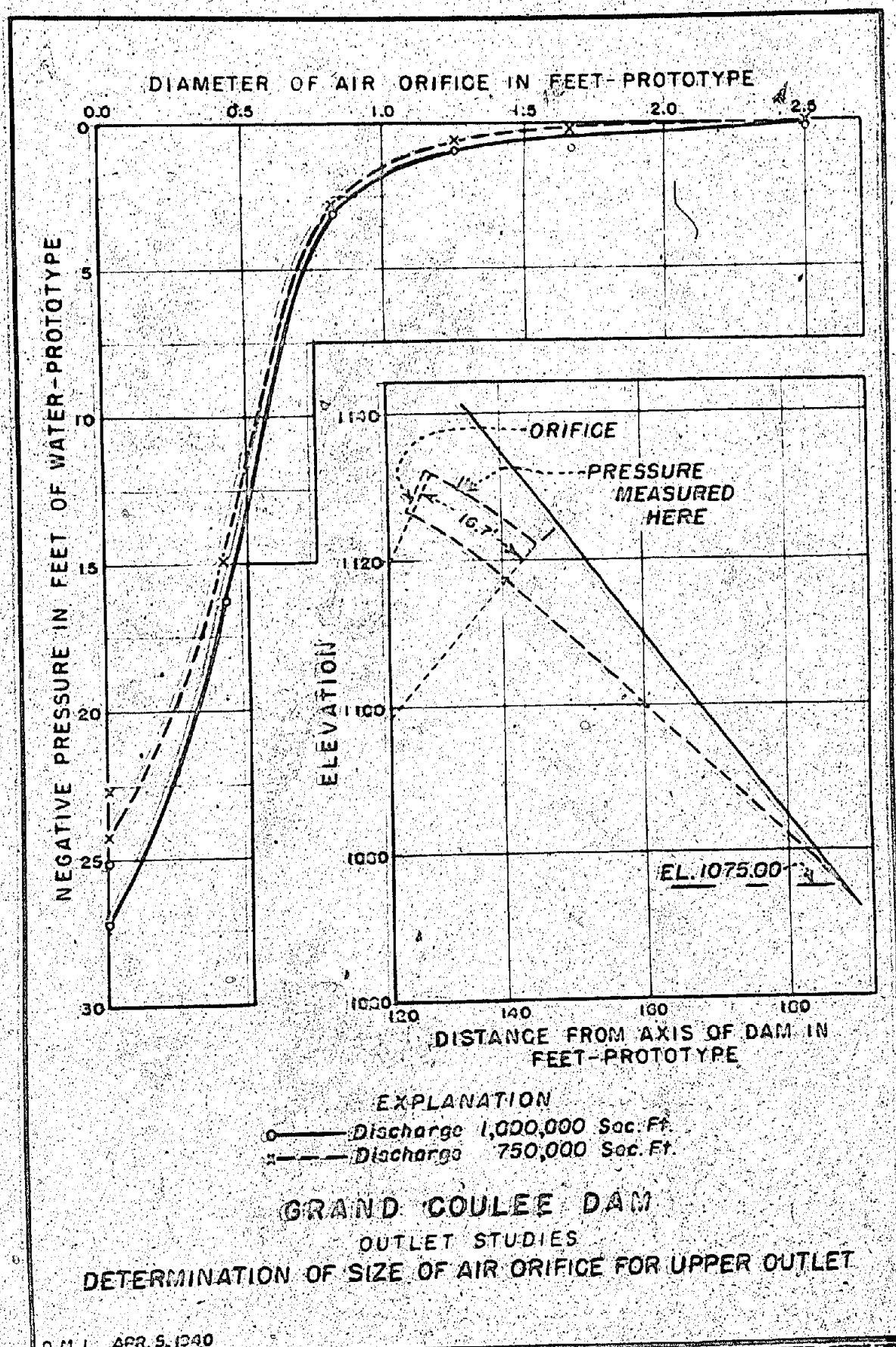


FIGURE 32. RELATION OF AIR ORIFICE DIAMETER TO PRESSURE IN UPPER OUTLET

quantity of air equivalent to 50 percent of its volume,[16] the amount of air required is:

$$\frac{4,[illegible]}{4}(0.50) = [illegible] \text{ second-feet}$$

Assume further, that a drop of pressure of 2 feet of water below that of atmospheric pressure exists in the conduit.

From the general gas law the quantity of free air required per second is given by:

$$\frac{PV}{T} = \frac{P_1V_1}{T_1}$$

Where, P = original pressure

P_1 = final pressure

V = original volume

V_1 = final volume

T = original absolute temperature

T_1 = final absolute temperature

Substituting the values:

$$\frac{14.[illegible]\,([illegible])}{[illegible] + [illegible]} = \frac{14.9\,V_1}{275 + 27}$$

V_1 = [illegible].2 cubic feet per second

The diameter of pipe necessary to pass this amount of air at atmospheric pressure is given by:[17]

$$d = 0.[illegible]7\left[\frac{Z}{p_1^2 - p_2^2}\right]^{1/5} V_a^{13/35}$$

Where, d = diameter of pipe in inches

L = length of pipe in feet

p_1 and p_2 = pressures in pounds per square inch

V_a = volume of free air flowing per minute

$$d = 0.[illegible] \left[\frac{[illegible]}{[illegible]} \right]^{1/5} \left[[illegible] \right]^{[illegible]}$$

$$d = 0.[illegible] ([illegible])^{1/5} ([illegible])^{[illegible]}$$

$$d = 0.[illegible] ([illegible]) ([illegible])$$

$$d = [illegible] \text{ inches}$$

A pipe of diameter [illegible] inches is equivalent to two pipes, each having a diameter of [illegible] inches if the difference in losses is neglected. Two [illegible]-inch pipes were used on the prototype.

A similar calculation for an upper outlet will show that two [illegible]-inch diameter pipes are required, while two [illegible]-inch diameter pipes were installed on the prototype.

Deflectors over Outlet Exits

At the time the studies on the new design of the outlets were started, the construction blocks in the Grand Coulee Dam had been poured to elevation [illegible] which made it necessary to place the outlet opening on the face of the dam above this elevation. The original thought was that the opening could be advantageously made longer had it not been for this limitation.

The final design for the intermediate outlet (Figure 30) which was developed under successive heats due to the contractor needing the drawings for construction purposes, did not have the deflectors over the outlets, although some thought had been given to the excessive spray due to water dropping into the channel of the outlet.

At a considerably later date, as the model of Shasta Dam spillway was completed, the outlets were installed using the design without the deflectors. Operation of the Shasta model showed clearly that some steps must be taken to minimize the spray from this cause. With the addition of the deflector over the outlet, the thickness of concrete between the nose and the face of the deflector was increased considerably beyond the minimum established at the beginning of the studies, as necessary for the placement of reinforcement steel. The point was raised as to the feasibility of taking advantage of these projections by moving the elbow and cone three feet downstream. This change moved the point of intersection between the invert of the channel and the face of the dam up the slope thirteen feet, thus appreciably reducing the exposed opening. In the meantime, construction had progressed such that this movement of the elbow and consequent shortened opening could not be used in the intermediate tier, but it was adopted for the upper tier in Grand Coulee Dam, and for all outlets in Shasta Dam.

The operation of the model of the Shasta Dam spillway showed that disturbances at the outlet exits prevailed at two flow conditions:

1. When the spillway is operating alone.
2. When the outlets are discharging simultaneously with a small quantity of water passing down the spillway.

The first condition is caused by the spillway flow dropping into the invert of the outlet channel and then being deflected into the air causing severe spray. Since the conduits in the upper tier at Grand Coulee Dam were placed directly above the corresponding ones in the lower tiers for economy in the tunnelback structure, a similar action took place at the intermediate tier when the upper ones were operating alone. The second condition is a result of the spillway flow intersecting the outlet jet. The frictional losses of a thin sheet of water flowing down the face of the spillway are greater than the corresponding losses in the outlet, and since the head causing the flow is the same in each case, the velocity of the outlet jet is much greater. This difference in velocities causes the disturbance at the intersection of the two jets. There is also a difference in the velocities of the two jets when the spillway is operating at maximum capacity, the spillway flow being the faster, but the thick sheet of the spillway is not conducive to the formation of spray. Since the spray released in the second condition is considerably less than that in the other, and furthermore, would seldom occur, no attempt was made to remedy the unfavorable action.

Location and Shape of Deflectors

The first condition was prevented by placing the spillway jet on a trajectory above the openings in the face of the dam. The

raised portion, or deflector, was parabolic in the direction of flow and blended into the face by reverse curves.

In the case of the intermediate tier, it was possible to extend the deflector over the upper portion of the opening, thereby lengthening the length of the trajectory, care being taken to prevent the outlet jet from striking the under side of the deflector. An excessive length of the trajectory caused disturbance when the deflected jet returned to the face of the dam, the hydraulic model was employed to determine the shape and location of the structure. Two of the trial designs are contained in figure 35 (designs A and B), while the final design is shown on figures 36 and 37.

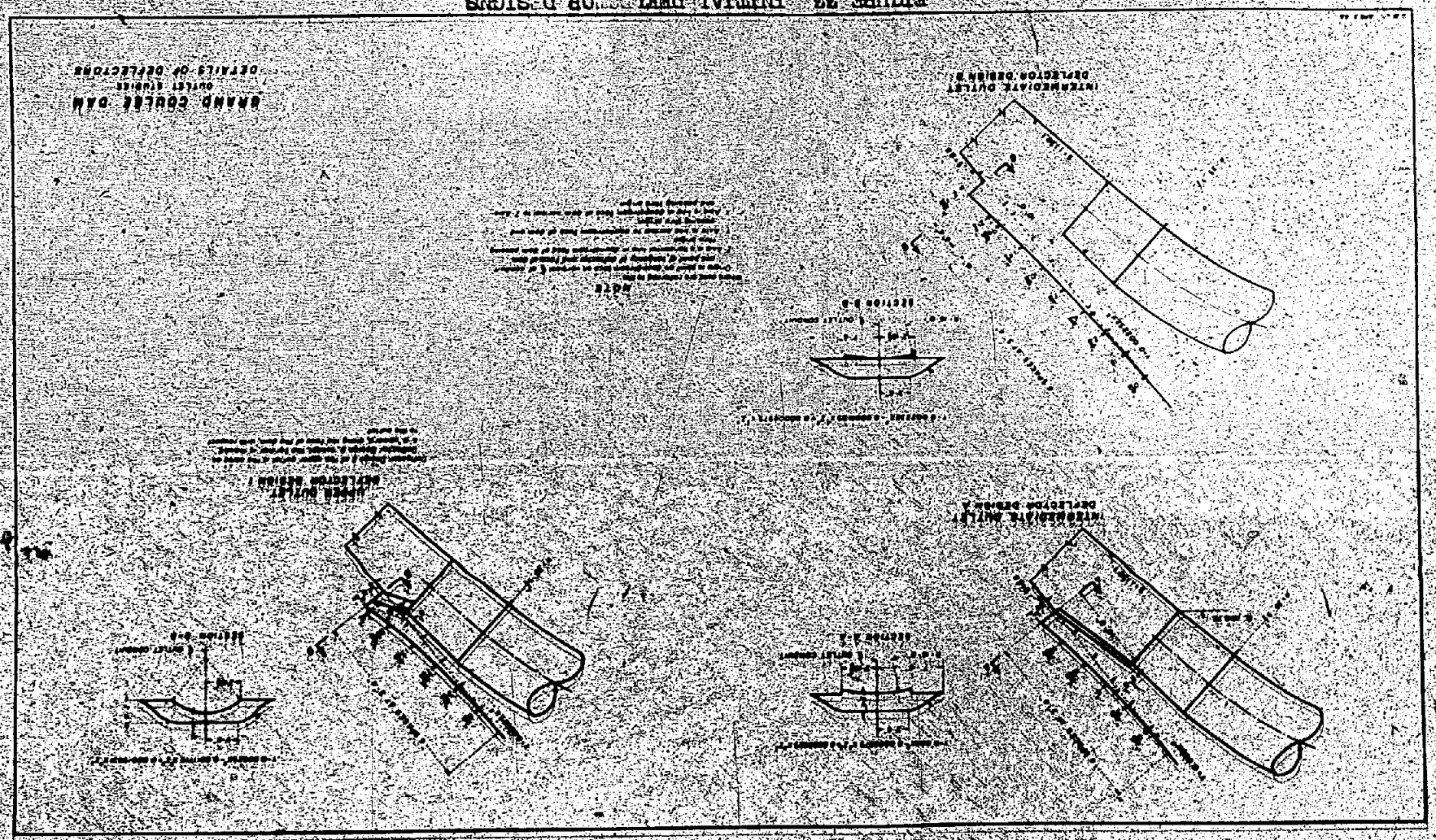
The under side of the projecting portion of the upper deflectors was arched to prevent interference with the outlet jets (figure 35, designs 1 and 2). Except for a change in the type of control gates, the final design outlet and deflector for the upper tier shown on figures 36 and 37 was adopted for all three tiers at Shasta Dam.

In these studies it was assumed that an abundance of air was available through the conduit to aerate the deflected portion of the spillway jet. Figure 38 illustrates the action of the deflectors when the jets were aerated and with the air supply pipe closed; although the figure pertains to Shasta Dam, the results are the same.

Comparisons of Prototype and Model Studies

To correlate the action on the prototype and the hydraulic model studies beyond the limits of visual observations, piezometer openings were placed in the lining of the right outlet of block

FIGURE 32. INITIAL DESIGNS FOR DAMS



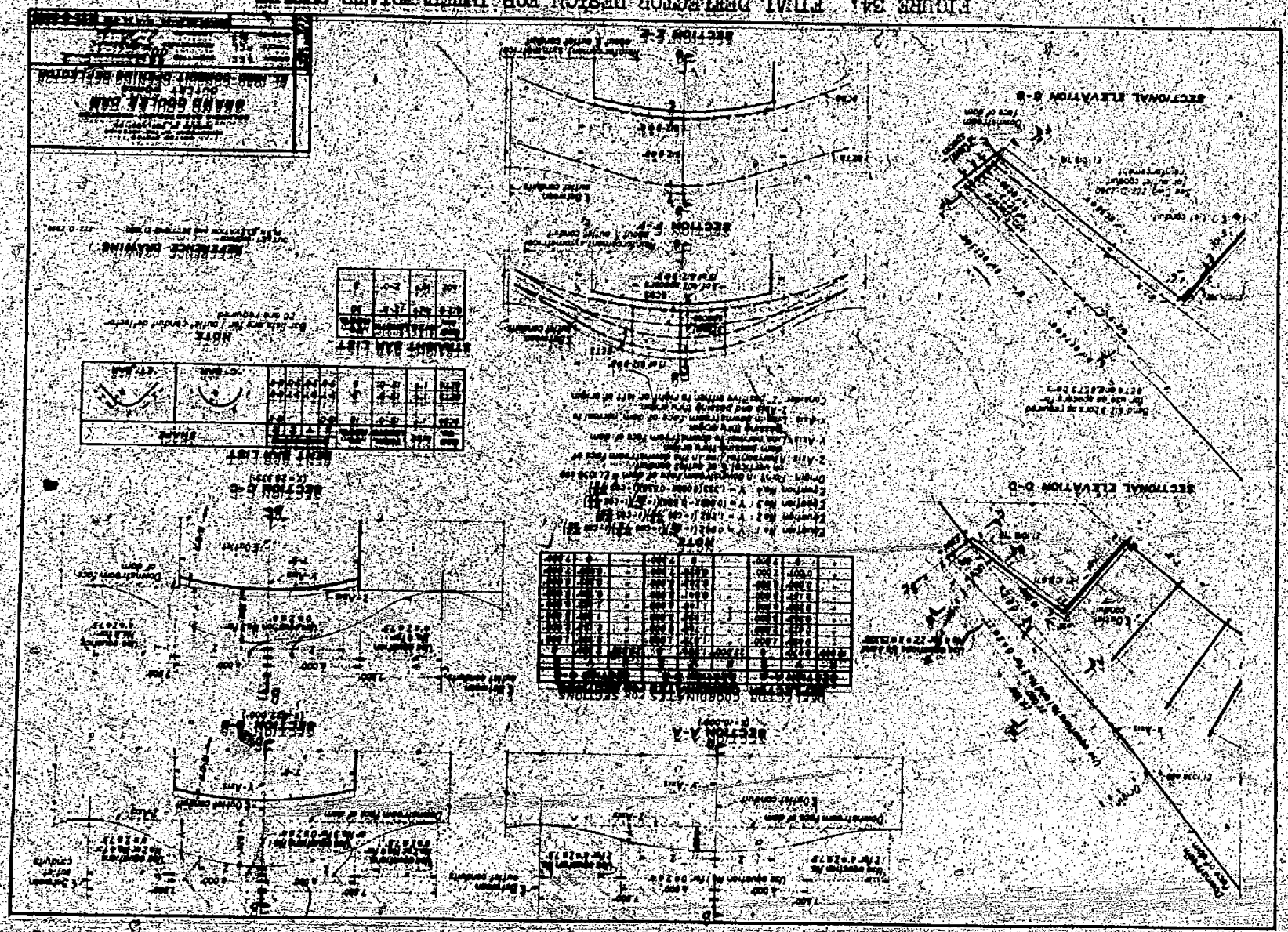


FIGURE 34: FINAL DEFLECTOR DESIGN FOR INTERMEDIATE OUTLET

DEFLECTORS AND AIR VENTS

HEAD ON CENTERLINE MOTIONLESS, RE:75 KNOT

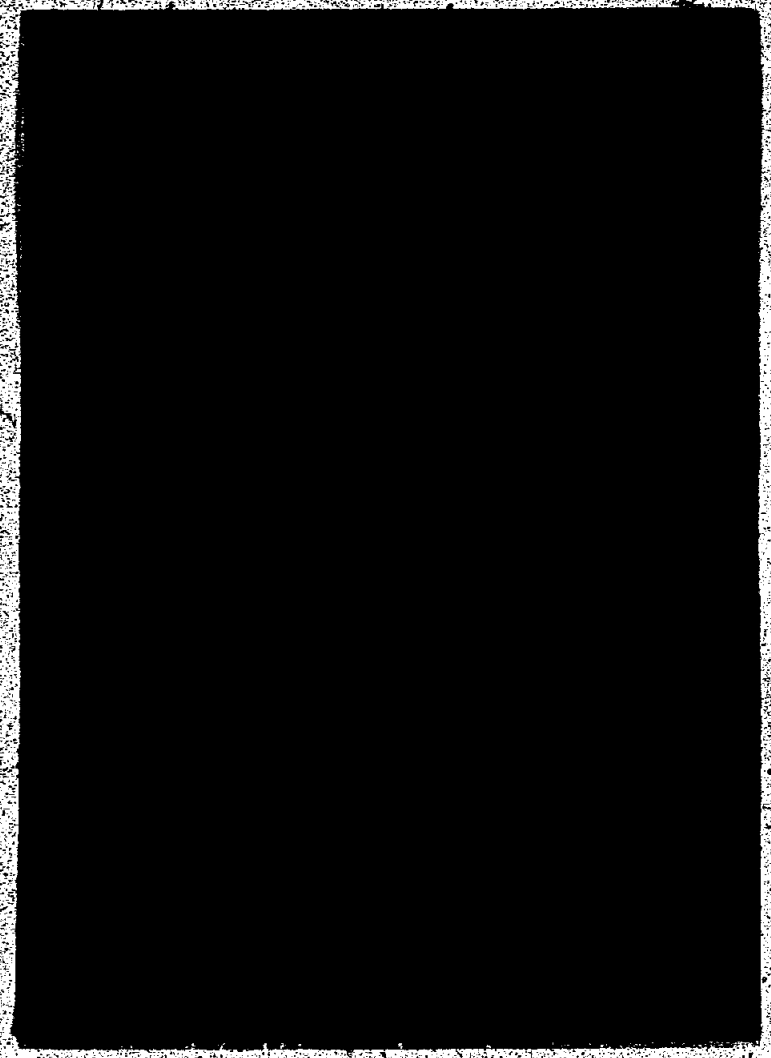


FIGURE 36. PROTOTYPE OUTLET VENTS = INTERMEDIATE TIER

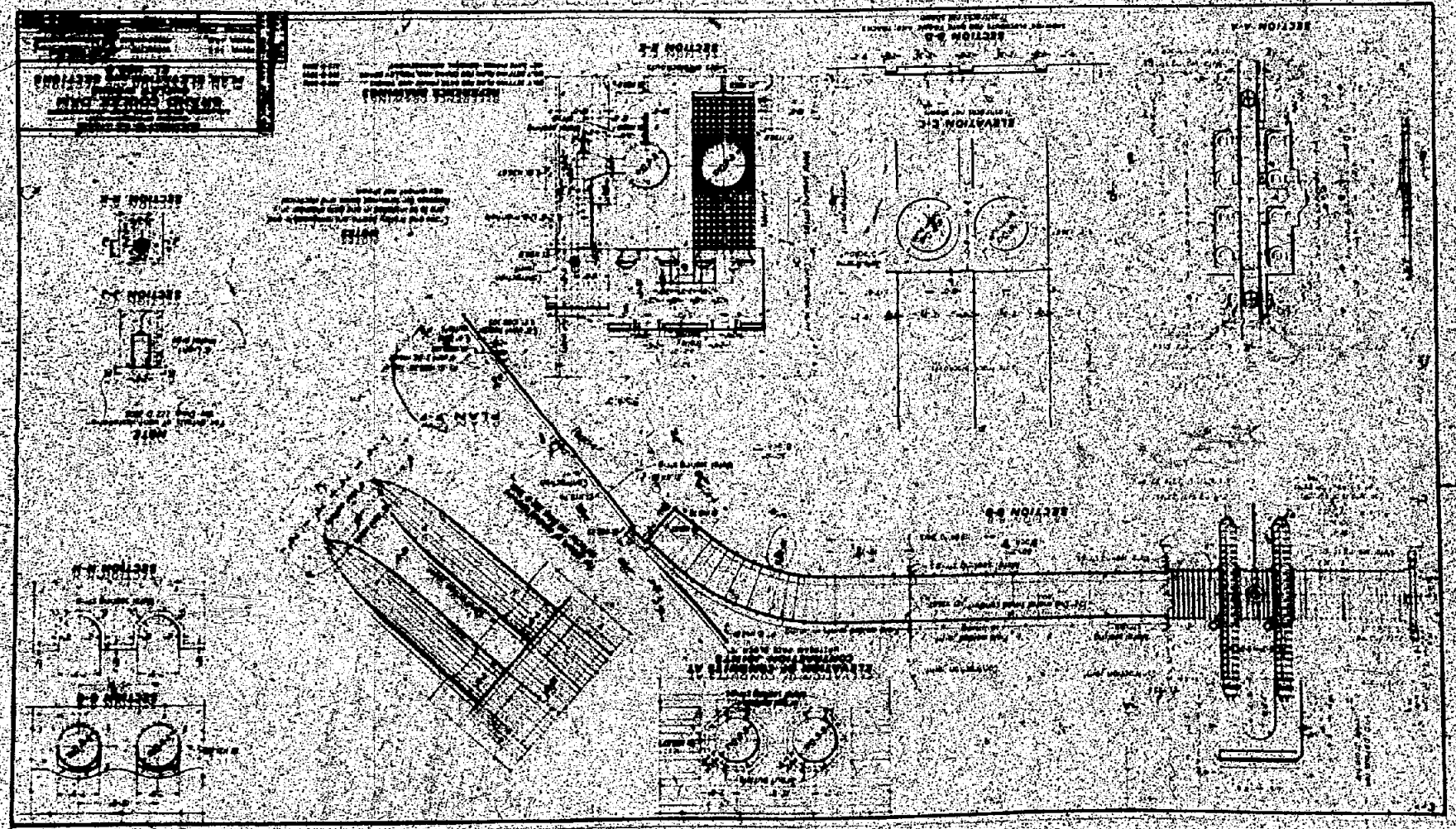


FIGURE 36. FINAL DESIGN OF UPPER OUTLET

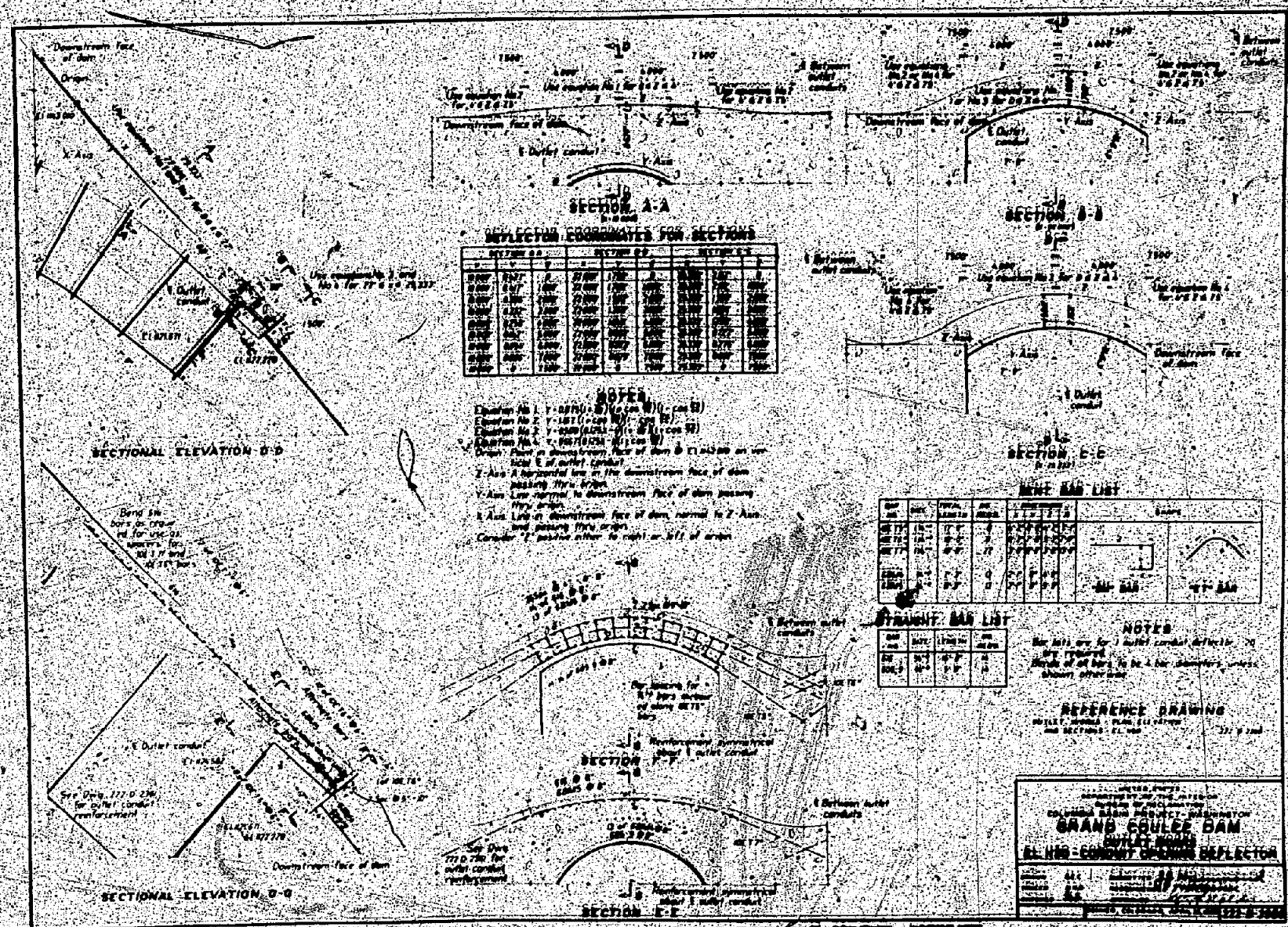
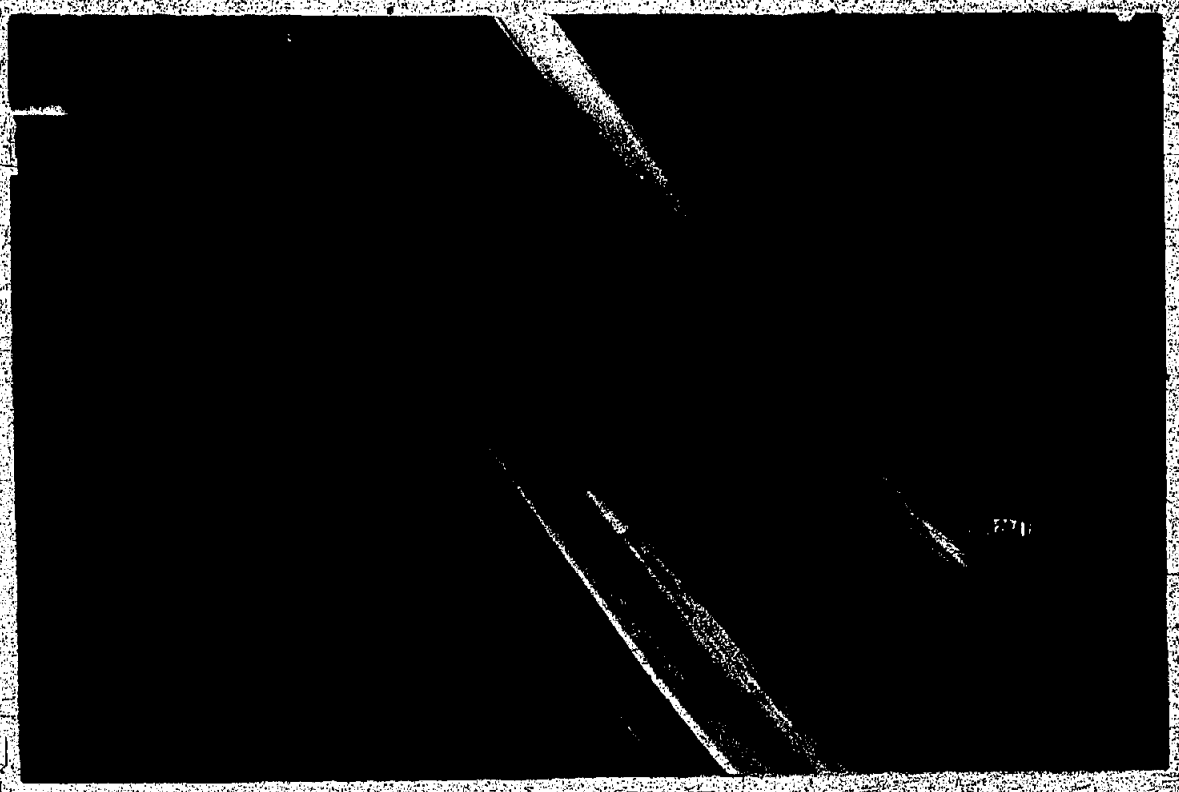
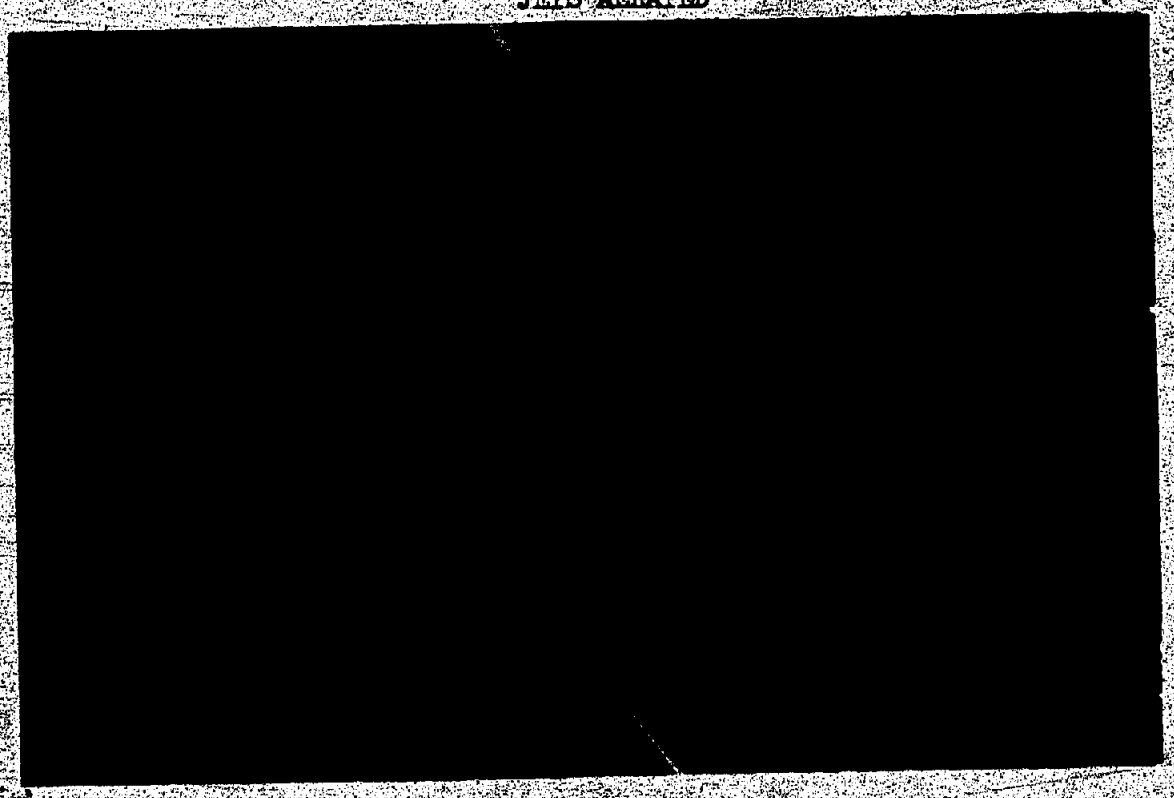


FIGURE 37. FINAL DEFLECTOR DESIGN FOR UPPER OUTLET



JETS AERATED



JETS NOT AERATED

SPILLWAY DISCHARGE 50,000 SECOND-FEET, DEFLECTORS IN PLACE

FIGURE 38 - EFFECT OF AIR CUTS ON DEFLECTORS

number 51 at elevation 1036.67 and 1136.67 (figures 39 and 40). A suitable piping system connect these openings to terminals in the respective galleries.

Since the outlets will not be operated under heads less than 40 feet after the completion of the dam it was necessary to study the flow conditions under low heads during the construction period. Field conditions have permitted the study of the intermediate tier operating at or below a head of 27.15 feet; additional studies will be made in the future.

Visual Observations of the Prototype

Visually, the action of the prototype jets from the intermediate tier corresponded to that of the model. The intake of air through the vents at the intersection of the apron with the face of the spillway substantiated the model studies. Although the action of the deflectors cannot be ascertained until the dam is completed, their location with respect to the jet issuing from the outlet was satisfactory in that there was ample space for the admission of air beneath the deflector. Consideration of the lower tier of outlets was limited to the action of the surface of the tailwater. The behavior in the field structure, so far as could be ascertained by visual observations, was strikingly similar to that in the 1:120 model.

Pressure Studies in Intermediate Outlet of Prototype

A comparison of the pressures in the intermediate outlet of the prototype and those determined on the model is shown on figure 41; the air vents at the ring-seal gates were closed in each case.

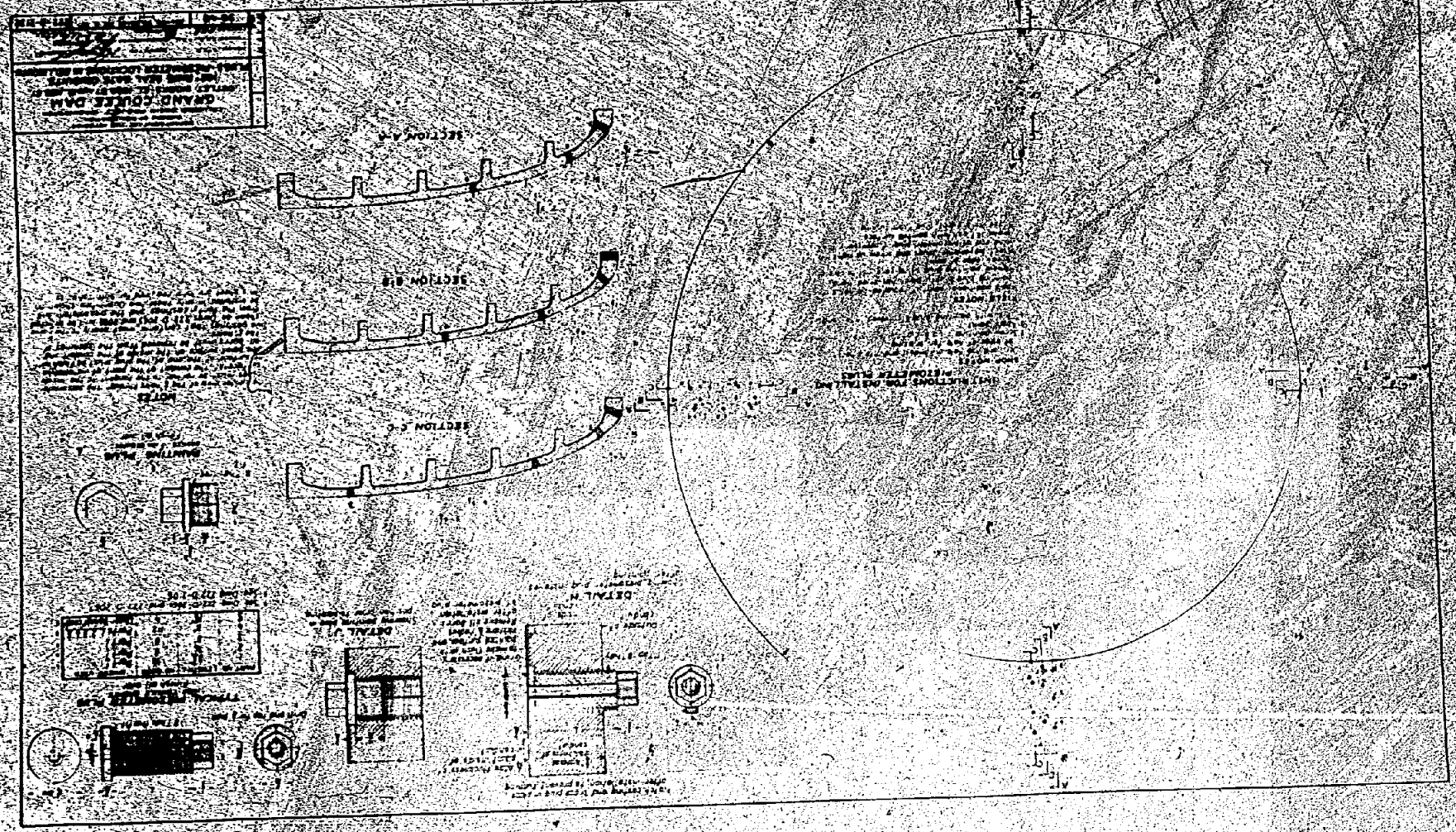
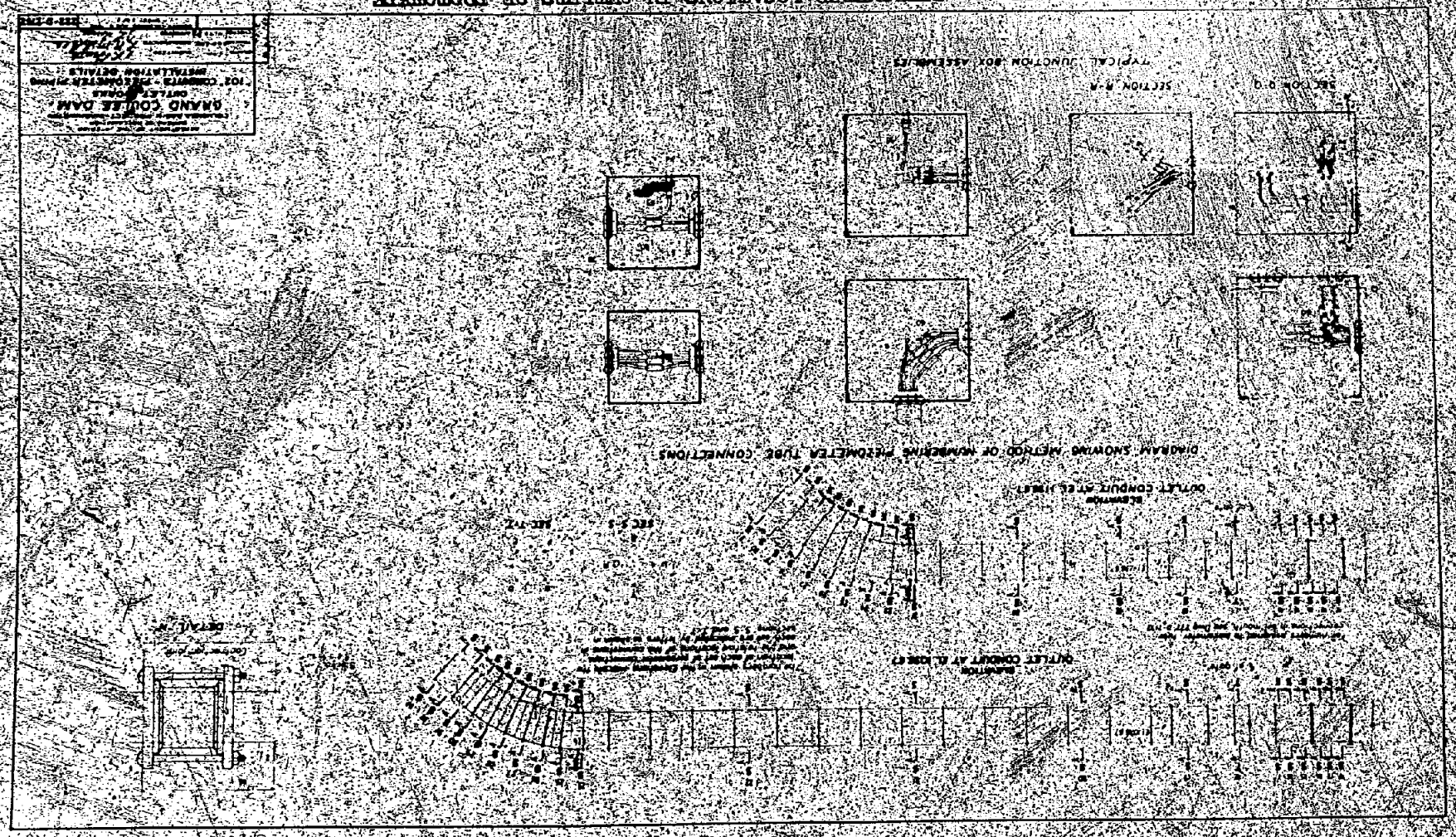


FIGURE 39. PIEZOMETER LOCATIONS IN BELLMOUTH OF PROTOTYPE

FIGURE 40. PIEZOMETER LOCATIONS IN OUTLETS OF PROTOTYPE



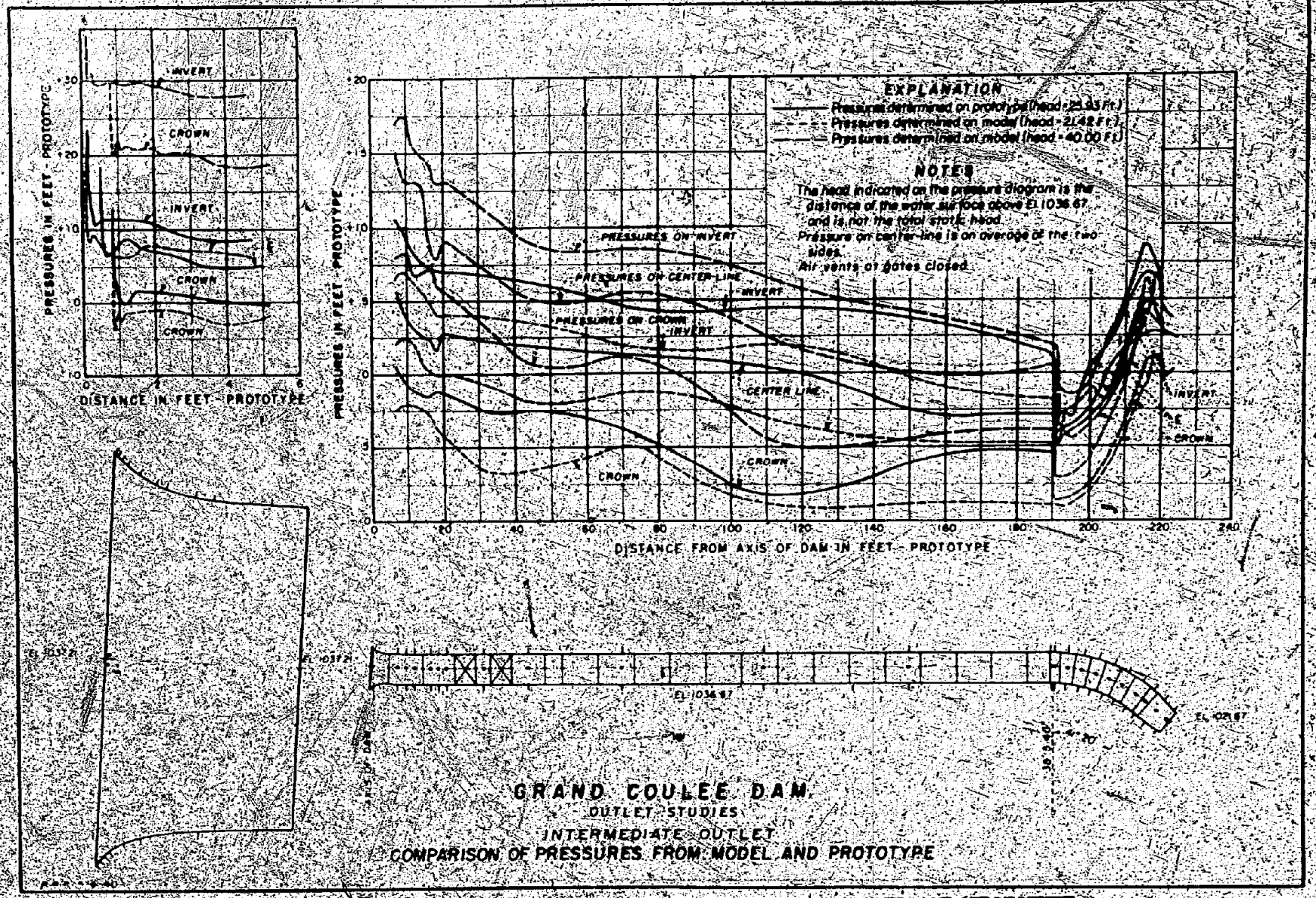


FIGURE 11. COMPARISON OF PRESSURES DETERMINED ON MODEL AND PROTOTYPE

The similarity of the pressure curves is apparent. These graphs indicate that the pressures on the prototype are slightly higher than those on the model when operating under the same head, but sufficient data have not been received to substantiate this deduction.

The effect of the intake of air into the conduit is shown on figure 42. When the air vents are open, the sub-atmospheric pressures along the crown of the conduit are relieved with air at a pressure slightly less than that of atmospheric pressure; hence, the conduit is not filled with water. This same condition occurred on the model for all heads below 50 feet on the centerline of the bellmouthed entrance.

The outlets will normally operate with the air vents at the gates open. Pressure curves for this condition are shown on figure 43. The inconsistency of the pressures on the invert at the downstream end is probably due to an air trap in the long tubes connecting the piezometer openings to the terminals; however, as a whole, the results are most gratifying.

Additional prototype data will be obtained when the field conditions are favorable, and the program will eventually include the study of the upper tier of outlets. The feasibility of such a program is irrefutable as such a correlation of data has never before been attempted.

Present Studies

Recent designs of river outlets by the Bureau of Reclamation have replaced the ring-follower type of slide gate by a tube valve.

FIGURE 42. EFFECT OF AIR ON PRESSURES DETERMINED ON PROTOTYPE

GRAND COULEE DAM
OUTLET STUDIES
INTERMEDIATE OUTLET
PROTOTYPE PRESSURES WITH AIR VENTS OPEN AND CLOSED

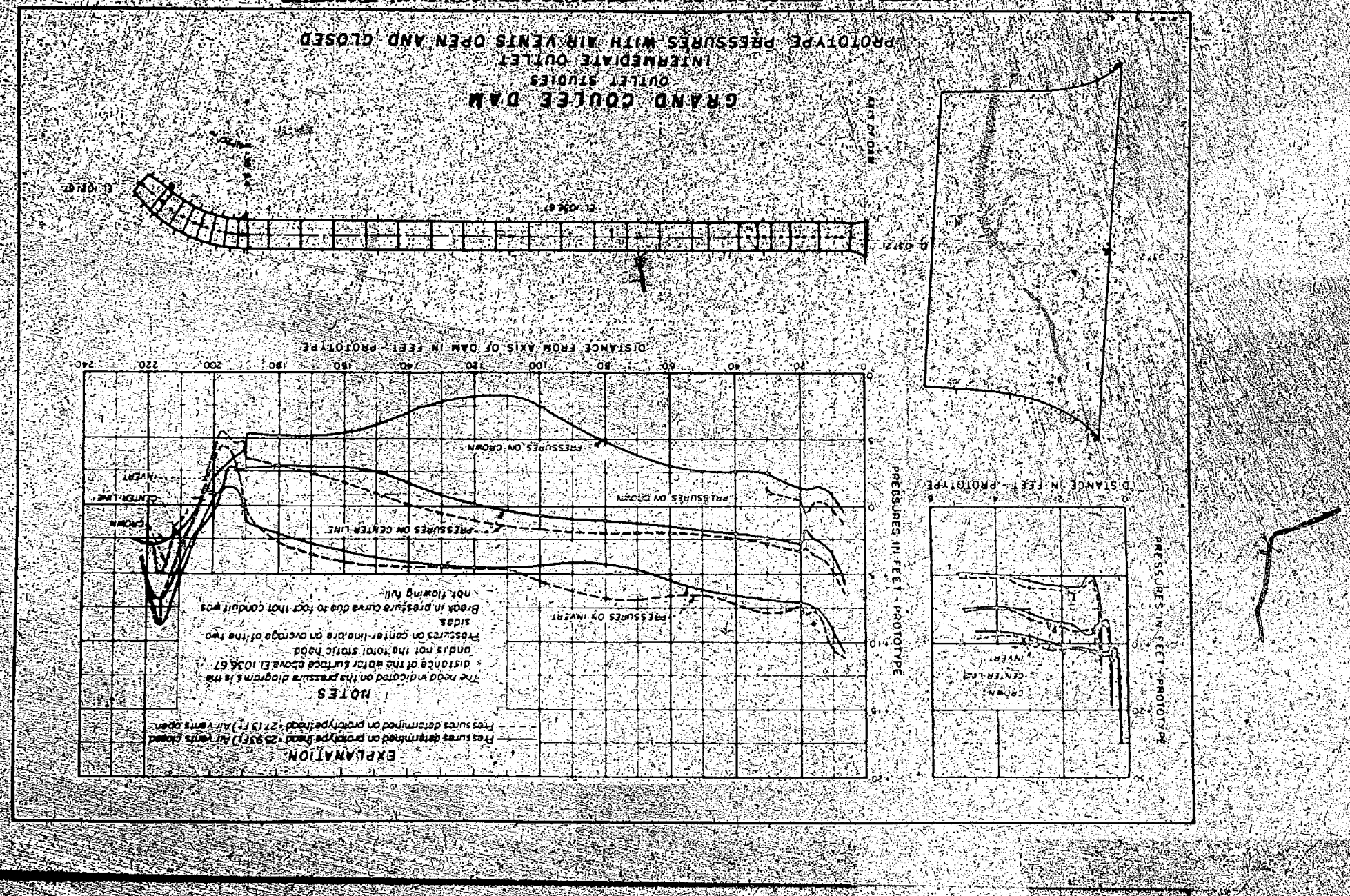
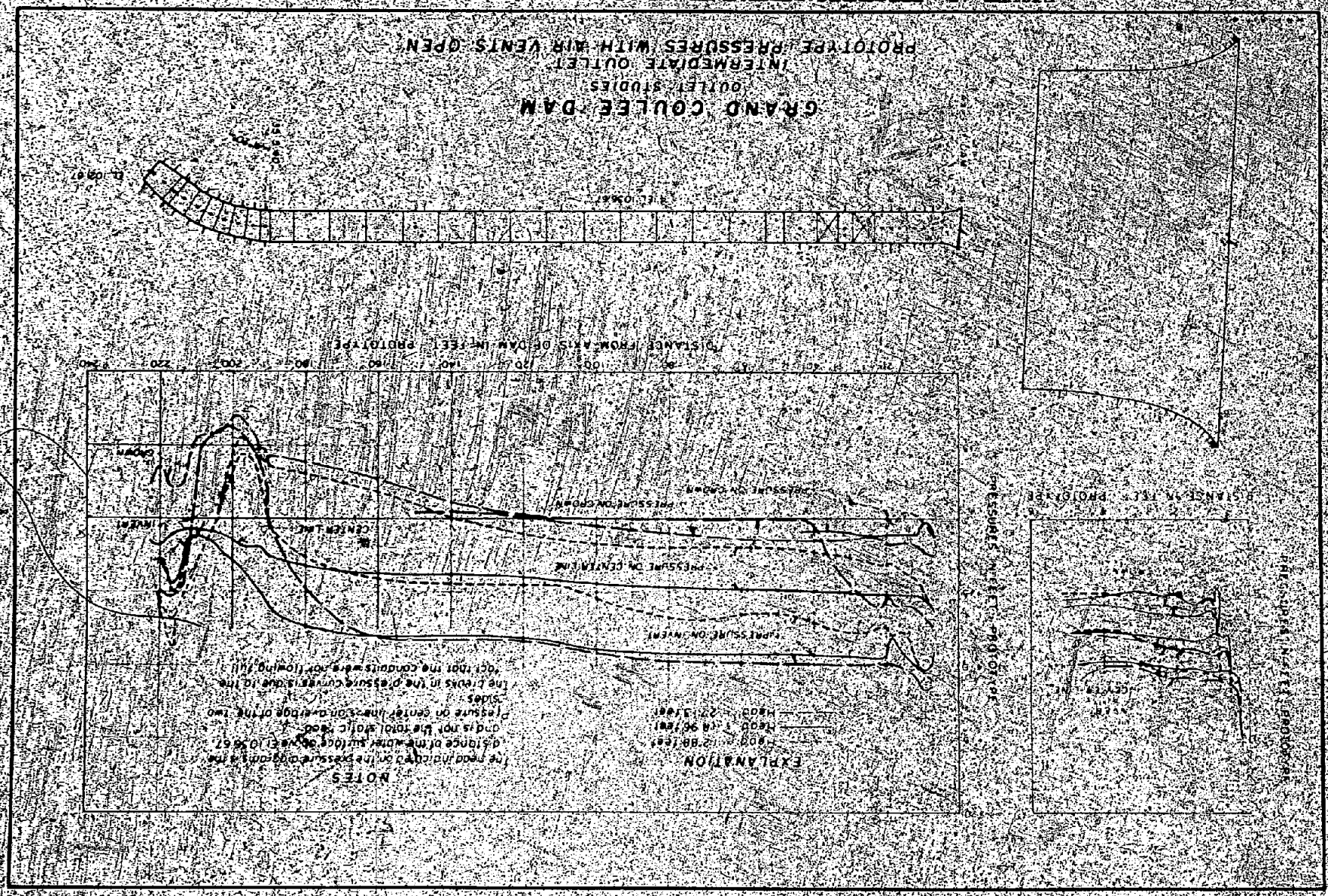


FIGURE 43.- PRESSURES DEVELOPED ON PROTOTYPE WITH AIR VENTS OPEN.



Several advantages are claimed for this latter type. First, the valve operating mechanism is less expensive, secondly, the valve opening may be controlled over an undetermined range resulting in closer regulation of the river flow, and thirdly, the allowable operating head on the valve is greatly increased.

Although the profile of the centerline of the conduit remains unchanged, studies have not progressed to a stage which will determine the shape of the valve section.

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APPENDIX

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