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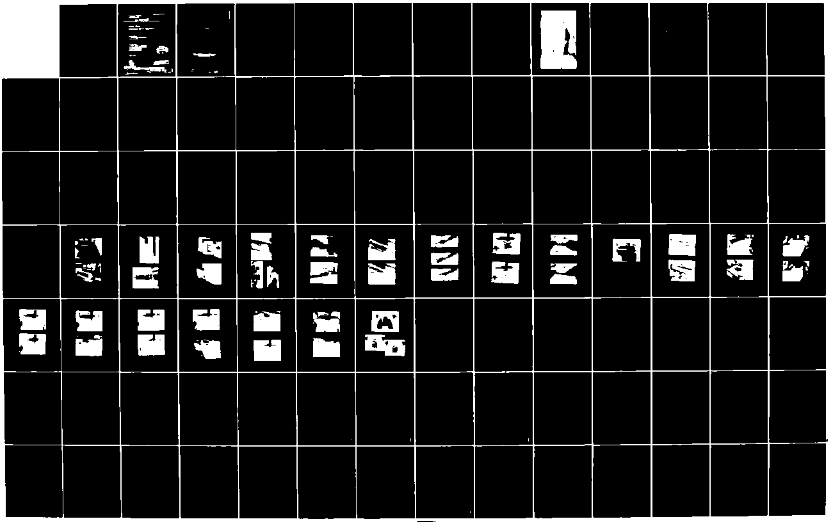

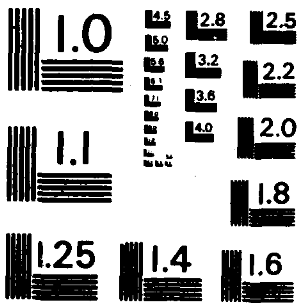

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Libby Dam, located on the Kootenai River 17 miles upstream from the town of Libby, Montana, rises about 422 feet above bedrock and includes a spillway with two 48-foot-wide bays, three 10- by 17-foot regulating sluices, and a powerhouse for eight 105-megawatt generators. At maximum pool, spillway capacity will be 145,000 cfs and regulating sluice capacity will be 61,000 cfs. The 116- by 300-foot stilling basin is designed for a maximum spillway flow of 50,000 cfs. Two separate 1:50-scale models were used to study the

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) spillway, regulating sluices, stilling basin, outlet channel, diversion, and proposals to reduce nitrogen levels downstream from the project. Alternative designs for bellmouthed intakes of the regulating sluices were studied in a 1:20-scale model. The model tests showed that the original spillway abutments, center pier, sluice intakes, spillway chute, and stilling basin should be revised. Satisfactory designs for these elements were developed. Deflectors, flip buckets, and combinations of the two failed to produce flow conditions that would reduce nitrogen levels downstream from the stilling basin for the expected range of sluice discharges during the first years of project operation. Sizes of rock needed to protect the banks and runout area below the stilling basin, discharge required to clean the basin of debris, bank outlines for an excavated exit channel, diversion procedures to use during second and third construction stages, and debris deflectors for legs of a temporary trestle were determined.

PREFACE

Hydraulic model studies for Libby Dam were authorized 21 December 1964 by the Office, Chief of Engineers, at the request of the U.S. Army Engineer District, Seattle. The studies were made at the North Pacific Division Hydraulic Laboratory, Bonneville, Oregon, under the supervision of Mr. A. J. Chanda, Chief of the Hydraulics Branch, and Mr. H. P. Theus, Director of the Laboratory.

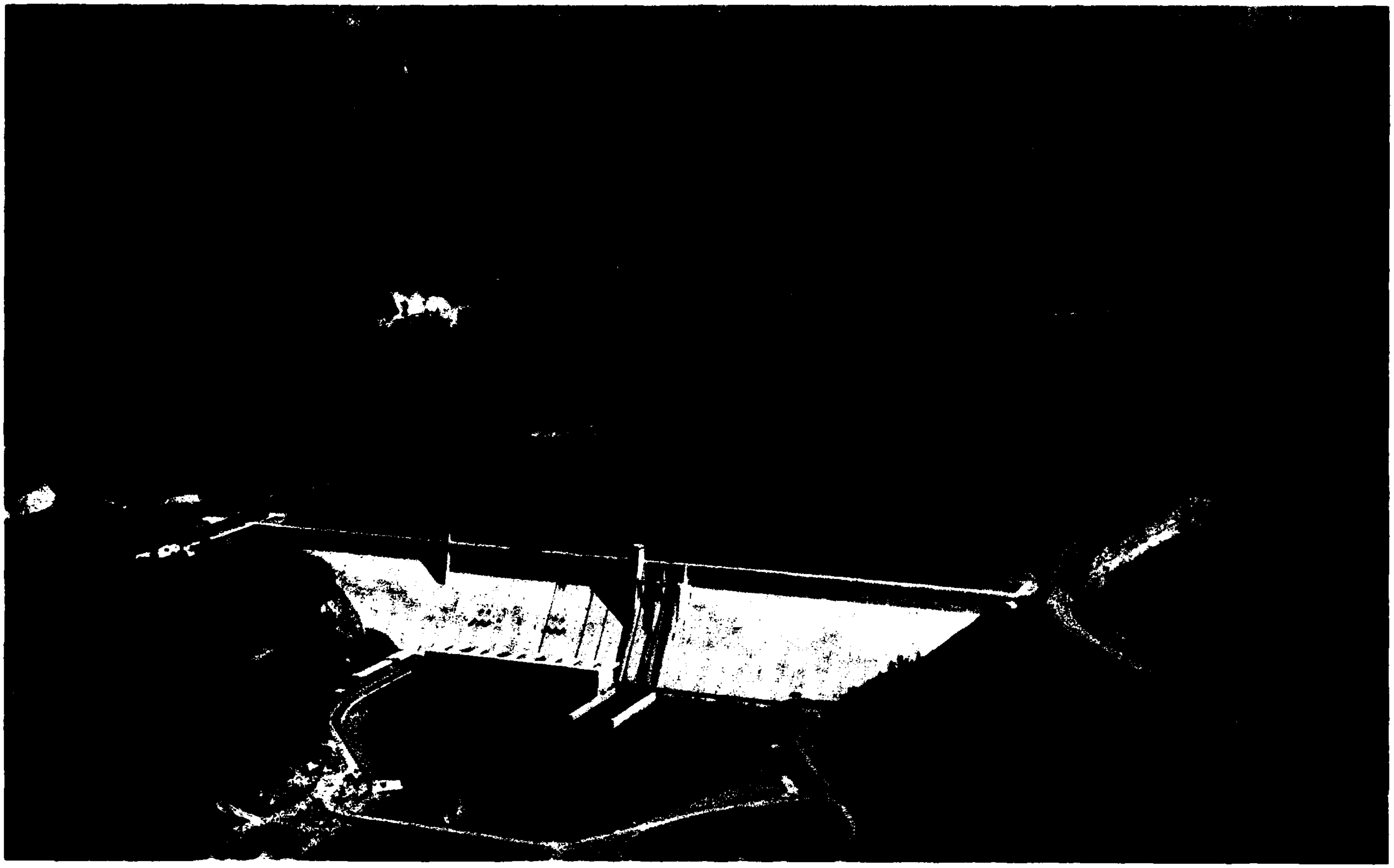
The spillway, stilling basin, excavated tailrace, and diversion stages were studies by Messrs. P. M. Smith, R. L. George, and D. E. Fox during the period September 1967 to July 1970. Mr. Smith tested plans for the sluice intakes from February 1967 to March 1970. Methods to alleviate nitrogen supersaturation were investigated by Mr. T. D. Edmister from July 1972 to March 1973. This report was prepared by Messrs. L. Z. Perkins and R. W. Parker and edited by Mr. Theus.

CONTENTS

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENTS

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimeters
feet	0.3048	metres
miles (U.S. statute)	1.609344	kilometres
acre-feet	1233.482	cubic metres
feet per second	0.3048	metres per second
cubic feet per second	0.0283168	cubic metres per second
pounds (force)	4.4482	newtons
pounds per square inch	0.0703	kilograms per square centimetre
kilowatts	60,000.0	joules per second



Aerial View of Libby Dam

LIBBY DAM, KOOTENAI RIVER, MONTANA
Hydraulic Model Investigations
PART I: INTRODUCTION

The Prototype

1. Libby Dam (frontispiece) is located on the Kootenai River 17 miles* upstream from the town of Libby in northwestern Montana. Figure 1 shows the vicinity map of the area. The project (plate 1) is a part of the comprehensive plan to develop the Columbia River basin in the interests of flood control, power generation, irrigation, recreation, and related water uses. Construction of the project by the U.S. Army Corps of Engineers began in the spring of 1966; control of the river at the dam began in March 1972.

2. The dam rises 422 feet above bedrock, is 2,955 feet long at the crest, and includes a powerhouse with an initial installation of four 105,000-kilowatt generators with provision for four additional units. The reservoir, Lake Koocanusa, has usable storage capacity of 4,965,000 acre-feet and backs water 42 miles inside Canada. The normal full and minimum pools are at elevation 2459 and 2287 at the dam.** This storage provides flood control for the Kootenai River valley and together with 15.5 million acre-feet of Canadian storage is a major factor in controlling floods along the Columbia River. The largest discharges of the Kootenai River at Libby, Montana, have been 130,000 cfs in 1894 and 121,000 cfs in 1916. The computed maximum flood outflow from the reservoir is 206,000 cfs with the pool at elevation 2459—145,000 cfs spillway flow and 61,000 cfs through the regulating sluices.

3. The spillway is an ogee-type concrete structure with crest at elevation 2405 (plate 2). The weir profile corresponds to the Corps of Engineers' high-dam shape for a design head of 46.4 feet, which

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* A table of factors for converting U.S. customary units to metric (SI) units of measurement is shown on page iii.

** All elevations in this report are in feet above mean sea level.

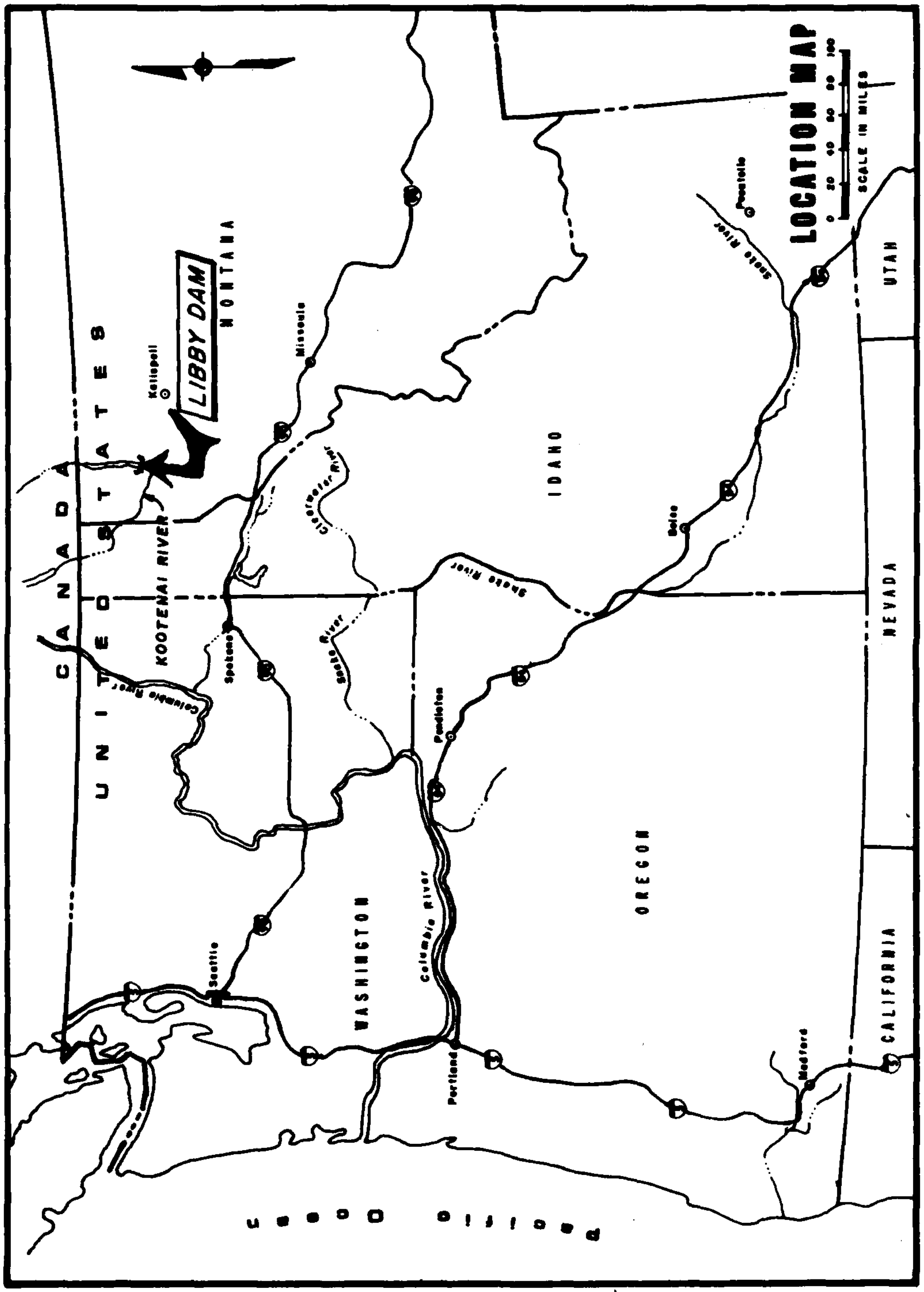
LOCATION MAP
0 20 40 60 80 100
SCALE IN MILES
LIBBY DAM
UNITED STATES
CANADA
MONTANA
IDAHO
WASHINGTON
OREGON
NEVADA
UTAH
CALIFORNIA
KOOTENAI RIVER
Columbia River
Snake River
Clearwater River
Kalispell
Missoula
Spokane
Pendleton
Boise
Pocatello
Seattle
Portland
Medford
Pacific Ocean

Figure 1

equals 85.7 percent of the maximum head of 54.0 feet. Flow is controlled by two 48-foot-wide by 59-foot-high tainter gates supported by elliptical abutment piers and a 20-foot-wide center pier with circular nose.

4. Three 10- by 17-foot sluices, sized for 35,000 cfs at minimum pool and 61,000 cfs at normal pool, pass through the spillway (plate 2). The sluice entrances have compound elliptical curves, and t outside sluices are skewed to discharge into the 116-foot-wide s 1-way stilling basin (plate 3).

5. Sluice discharge is regulated by hydraulically operated tainter valves. Downstream from the valves, the invert curve follows the theoretical trajectory for the full-pool head of 264.5 feet. At full pool and full valve opening the sluices have a computed freeboard of 2.5 feet. Emergency closure is by tractor-type gates that operate in slots about 14 feet downstream from the axis of the dam. Downstream from the emergency gate slots, the sluices are lined with steel for approximately 43 feet on the bottom and sides and for 5.5 feet on the roof.

6. The stilling basin (plate 2) is a hydraulic jump-type energy dissipator 116 feet wide and 300 feet long with horizontal floor at elevation 2073 and a 12-foot-high sloping end sill. The basin will provide satisfactory dissipation of energy for a flow of 50,000 cfs over the spillway when an additional 10,000 cfs passes through the powerhouse. With spillway flows greater than 50,000 cfs and up to 90,000 cfs, a part of the hydraulic jump and energy dissipation will be outside the stilling basin over the rock-protected runout channel. At approximately 90,000 cfs the jump will be completely swept from tne basin and flip bucket action will plunge the flow into the runout channel. Greater spillway flows will be flipped into the unprotected channel beyond the runout. Since rock at the toe of the dam is sound, a rare flood that would sweep the jump out of the stilling basin will not endanger the structure. The top of the training walls at elevation 2142 provides about 2 feet of freeboard for the stilling basin design discharge. Overtopping due to surge action is acceptable. Downstream from the end sill, a runout channel sloped 1V on 6H extends from approximately elevation 2080 to the excavated tailrace at elevation 2110.

Need for Model Studies

7. Model analysis of the Libby spillway and regulating sluices was required in the interests of safety, economy, and development of a competent design. The primary purpose of the study was to check crest and pier pressures, head discharge relationships, eyebrows at sluice

outlets, abutment and pier shapes, training wall heights, and capacity of the stilling basin. As design and construction of the project progressed, model studies of tailrace excavation, riprap sizes, protection of stilling basin runout slope, diversion stages, flow deflectors to reduce nitrogen supersaturation, sluice intakes, flow required to clean stilling basin, stilling basin ejector drains, selective withdrawal structure, and tests with a density-stratified reservoir were made. Tests of the selective withdrawal structure with a density-stratified reservoir were published in a separate report.*

The Models

8. Three models were used in the studies. A portion of the forebay, the spillway, sluices, and powerhouse, and 1,600 feet of exit channel were reproduced in a 1:50-scale model for general study of the spillway, exit channel, and diversion during construction (photograph 1 and plate 4). A separate 1:50-scale model was used to study methods to reduce high levels of nitrogen in flow downstream from the project. This model included the spillway, sluices, stilling basin with runout slope, and a 200-foot-wide by 400-foot-long section of downstream channel (photograph 2 and plate 5). In both models all open-channel sections of sluices downstream from the valves were modeled. The intake of the right regulating sluice was reproduced in a 1:20-scale model (plate 6). The model reproduced a portion of the forebay and upstream face of the dam and included the intake bellmouth, emergency gate slots, and valve section; the valve was not included. The model forebay consisted of two closed steel tanks that allowed the full head of 247.7 feet (12.4-foot model) on the intake to be reproduced. The test bellmouths were precision castings of epoxy material (photograph 3).

* Smith, P. M., A. G. Nissila, "Selective Withdrawal System, Libby Dam, Kootenai River, Montana, Hydraulic Model Investigation," Technical Report No. 125-2, Dec 75, U.S. Army Corps of Engineers, North Pacific Division Hydraulic Laboratory, Bonneville, OR.

9. Water used in the models was supplied by pumps, measured by meters in the supply lines, and diffused into adequate forebay areas upstream from the respective test sections. Tailwater elevations were controlled by adjustable overflow weirs according to computed rating curves furnished by the Seattle District, U. S. Army Corps of Engineers (plate 7). Standard laboratory instruments and procedures were used to measure pressures, velocities, current directions, and water surface elevations in the models. Structures were built of wood and plastic. Channel topography was reproduced in concrete grout or built of plywood. Crushed rock with a specific gravity of 2.74 was used to represent various sizes of riprap in the general spillway model. Coarse sand and 3/8-inch-minus gravel were used to represent gravel and rock up to 200 pounds during tests to determine the self-cleaning ability of the stilling basin.

10. Model measurements were converted to prototype values with equations of similitude based on the Froude model law. Pressure lower than -33 feet of water indicated vaporization in the prototype and had no other significance except to show relative pressure conditions in the model.

PART II: TESTS OF COMPLETED STRUCTURES

Plan A Spillway (Original Design), Plans A and B Stilling Basin

Spillway Crest and Chute

11. Details of the original structures are shown on plates 4, 8, and 9. The locations of piezometers in the crest, right abutment, pier, end sill, and sluice outlets are shown on plate 10. Initial tests were made with the exit channel at elevation 2097, which at the time was the proposed ultimate excavation. Free flows of 20,000 and 145,000 cfs and gated discharges of 20,000 cfs (6-foot opening at forebay elevation 2457) and 40,000 cfs (15-foot opening at forebay elevation 2443) were reproduced.

12. Although flow did not impinge on the bridge across the spillway or on the gate trunnions during the 145,000-cfs discharge, waves caused by flow around the abutments overtopped the upstream ends of the chute walls (photograph 4 and plate 44). Water deflected by eyebrows over the outside sluice outlets also overtopped the walls, and a rooster tail formed by converging flow downstream from the pier impinged on the bridge across the lower end of the chute (plate 12). Vortexes formed near both abutments when the gates were used to control high flows. Spray that overtopped the side walls at a gated flow of 20,000 cfs was no problem at 40,000 cfs.

13. Pressures at the piezometer locations shown on plate 10 are listed in table A for the spillway design discharge. The lowest pressures on the crest were -3 feet at piezometer C-20 near the centerline of the right bay and -7 feet at piezometer C-35 adjacent to the pier. Computed pressures at these locations were -1.5 and -14.5 feet, respectively. Minimum pressures on the pier and abutments were -12 feet (piezometer P-8) and -6 feet (piezometer A-8) and occurred just downstream from the stoplog slots. Pressures in the sluice outlets indicated that the chute flow impinged on the sluice floors near their intersection with the chute (pressures of 51, 52, and 45 feet for piezometers E-6, E-8, and E-14, respectively).

14. Additional piezometers were installed in the sluice outlets and pressures were measured with and without eyebrows above the outlets with sluice flow only, spillway flow only, and combined flow. Maximum impact pressures with and without the eyebrows were 111 and 135 feet in the spillway bucket and on the floors of the side sluices (piezometers E-19 and E-7) when the sluices were closed and 145,000 cfs passed over the spillway. No areas of low pressures were observed downstream from the impact points. With a combined sluice flow of 61,000 cfs and spillway flow of 145,000 cfs, the impact pressures decreased to 92 and 82 feet, respectively. The lowest pressure of -1 foot was at piezometers 2 and 10 behind the eyebrows. Above-atmospheric pressures in the air pockets beneath the spillway flow at the sluice outlets did not reduce sluice discharge during the project design flood of 206,000 cfs because the discharge was controlled by valves that were vented to maintain atmospheric pressure downstream from them. Removing the eyebrows eliminated overtopping of the lower chute walls by spillway flow, but overtopping still occurred with the combined sluice and spillway discharge of 206,000 cfs.

Plan A-Stilling Basin

15. The capacity of the Plan A stilling basin was inadequate for the design flow of 50,000 cfs with tailwater for 60,000 cfs. The minimum tailwater required to maintain a hydraulic jump in the basin is shown on plate 13. With flows to 10,000 cfs, the downstream channel (elevation 2097) produced a tailwater higher than necessary to maintain a jump. The jump could not be swept out by lowering tailwater. The maximum discharges at which a jump would remain in the stilling basin with available tailwater were 27,000-cfs spillway flow and 34,000-cfs sluice flow. Better flow distribution and mixing action caused the higher basin capacity with the sluice flow. The maximum capacity of the basin with sluice flow and raised tailwater was 40,000 cfs. Additional tailwater depth did not increase the basin capacity with spillway flow.

Plan B-Stilling Basin

16. Details of the Plan B stilling basin, which was 107.2 feet longer (4.2 D_2) than Plan A, are shown on plate 14. The maximum capacity of the stilling basin was 39,000 cfs with normal tailwater for that discharge and 44,000 cfs with tailwater for a combined spillway and powerhouse flow of 54,000 cfs (plate 15). Approximately 3 feet of additional tailwater were required for 50,000-cfs flow with 10,000-cfs powerhouse flow. Use of a vertical end sill had little effect on the basin capacity. Flow conditions with the project design flood of 206,000 cfs are shown in photograph 5. With tailwater for 206,000 cfs (elevation 2144) the water surface in the tailrace was at approximately elevation 2109, and waves covered the base of a proposed observation platform on the right bank.

Plan B Spillway, Plans C to D-1 Stilling Basin

Spillway Crest and Chute

17. The Plan B spillway (plate 16) had the same crest shape, bay widths, and tainter gates as Plan A (plate 8); but the pier and abutment lengths were reduced 34.1 feet, the upstream overhangs and the stoplog slots were removed, the abutment noses were changed from circular to elliptical, the upstream width of the pier was reduced to 20 feet, the chute width was changed to 116 feet, and the stilling basin was lengthened 40 feet and lowered to elevation 2059 (plate 17). Piezometer locations in the crest and right abutment are shown on plate 18. No piezometers were installed in the Plan B pier.

18. The design flow of 145,000 cfs passed over the revised spillway with a total head of 52.9 feet—1.0 foot lower than with the original design. Water surface and pressure profiles for 60,000 cfs and 145,000 cfs are shown on plates 19 and 20, respectively. The area of negative pressures on the crest increased; however, the lowest pressure on the crest at the design discharge was only -2 feet (piezometers C-20 and C-35, table B). With a gated flow of 60,000 cfs, pressures in the same areas were -3 feet. Pressures on the right abutment were positive with a free flow of 50,000 cfs (stilling basin

design discharge) and a head on the crest of 27.8 feet. With the spillway design flow of 145,000 cfs, pressures ranged from 19 feet at piezometer A-21 to -16 feet at piezometer A-26. The low pressure was indicative of possible cavitation in the prototype. In contrast, no negative pressures occurred on the circular noses of the original abutments which overhung the upstream face of the dam 5 feet and were subjected to lower velocities.

19. The 15-foot-high chute walls did not contain all the flow. At a gate opening of 18 feet (60,000 cfs), the upstream ends of the walls would have to be raised to elevation 2380 (27.5 feet high) to contain the rideup (plate 19). With a free flow of 145,000 cfs, the nappe was 6 feet deeper than the wall height (plate 20). Five designs of abutment deflectors—four to deflect flow (plans B to E) and one to deflect rideup only (plan F)—were investigated (plate 21). The spray was minimized and rideup on the wall was decreased by a 1-foot-wide deflector with 90-degree leading edge. The Plan F deflector for rideup only was the simplest and most effective but required higher chute walls. If the lower chute walls were desirable for structural reasons, the Plan E deflectors with capped walls would be adequate.

20. Because the Plan B pier was shorter than Plan A, the rooster tail caused by expansion of the nappes formed about 50 feet farther up the chute (elevation 2290), was higher, and extended farther downstream. The larger rooster tail and new location of the bridge over the stilling basin, which was at a lower elevation and farther upstream, reduced the spillway flow at which impingement occurred on the leading edge of the bridge to 60,000 cfs and on the bridge deck to 65,000 cfs. This condition was improved by attaching a deflector to the downstream edges of the pier (plate 22). Of the four deflectors tested, the 1-foot-wide, 45-degree deflector stabilized the rooster tail so that it stayed in the stilling basin instead of fluctuating beyond the training wall on the left and to the powerhouse on the right. The water surface profiles shown on plate 23 indicated that the pier deflectors had little or no effect on flow distribution in the stilling basin.

Pier Extension Plans A and A-1

21. The pier extensions shown on plate 24 were tested to determine whether they would reduce the size of the rooster tail and impact of flow on the stilling basin bridge. Plans A and A-1 were similar except that the height of the latter was reduced to 22 feet to match that of the chute walls. With the Plan A extension, impact on the elevation 2148 bridge deck occurred intermittently during gated flows greater than 100,000 cfs (gates open 30 feet or more). At large gate openings when the flow was in the unstable stage between gated and free flow, vortexes at the pier sometimes entrained air that enlarged the rooster tail. With free flow of 145,000 cfs, the low rooster tail usually passed underneath the bridge but occasionally impinged on the leading edge when minor fluctuations at the spillway crest caused the jets to swing from side to side.

22. The Plan A-1 pier extension was not overtopped by a free flow of 145,000 cfs (photograph 6). The upstream corner of the extension was occasionally overtopped during a gated flow of 110,000 cfs. The overtopping flow followed the narrow edge of the pier extension to about station 51+15 and did not spill into the center void. This plan was adopted for use in the prototype.

Discharge Rating, Plan B Spillway, Plan A-1 Pier Extension

23. Discharge rating curves for the Plan B spillway are shown on plate 25. The spillway design flood of 145,000 cfs passed through the spillway as free flow at pool elevation 2458. The free flow discharge with maximum operating pool elevation 2549 was 150,000 cfs.

Plan C Stilling Basin

24. Following tests of the 120-foot-wide by 228.78-foot-long Plan B stilling basin, the width was reduced to 116 feet, the length was increased 40 feet, and the basin was lowered 15 feet to elevation 2059. Details of the Plan C design are shown on plate 17. The stilling basin design discharge was increased from 50,000 cfs with tailwater for 60,000 cfs to 60,000-cfs spillway flow with the same tailwater.

Minimum tailwater required to confine energy dissipation of spillway flows within the stilling basin is shown on plate 26. Violent boils over the downstream runout slope occurred when the tailwater was 1 or 2 feet lower than shown. With normal tailwater (channel elevation 2097), the hydraulic jump at 60,000 cfs was very turbulent. Bottom velocities near the top of the runout slope were 10 to 14 fps, and waves were 6 feet high at the powerhouse and right bank observation platform. Flow conditions with a vertical end sill were similar.

Plans D and D-1 (Recommended Plan) Stilling Basin

25. For the Plan D stilling basin, the design criteria reverted to the conditions of Plans A and B; i.e., 50,000-cfs flow with 60,000-cfs tailwater. Plan D was a modification of Plan C with the floor raised from elevation 2059 to elevation 2073 and the length decreased from 268.78 to 248.78 feet. The width of the basin (116 feet) and the tops of the training walls (elevation 2135) were the same as Plan C (plate 29). In Plan D-1 the walls were raised to elevation 2142 (plate 32). These basins were tested with the tailrace channel at elevation 2110.

26. The minimum tailwater elevation required to confine the hydraulic jumps within the stilling basin for spillway and sluice flows are shown on plate 27. The capacity of the basin was 51,000 cfs with spillway flow only (photograph 7) and greater than the maximum sluice capacity of 61,000 cfs. Flow conditions with the design discharge of 50,000 cfs and tailwater for 60,000 cfs are also shown in photograph 7. Upwelling overtopped the Plan D training walls at elevation 2,135. Raising the walls 7 feet to elevation 2142 in Plan D-1 (final design) was recommended (photograph 8). When the spillway discharge was increased beyond 51,000 cfs, more of the hydraulic jump extended into the rock-protected runout channel until the jump was swept from the basin at approximately 90,000 cfs. A high level of energy was dissipated in the runout channel and higher waves were generated downstream (photograph 9). When the jump was swept out, the basin functioned as a flip bucket. With discharges of 90,000 to

100,000 cfs the flow plunged into the runout channel (photograph 10). At greater discharges the flow flipped into the unprotected channel downstream (photograph 11). The rare high discharges would erode the channel but should not endanger the structures.

Downstream Channel

Design Considerations

27. One of the problems that arose during the design of Libby Dam was the selection of an acceptable plan for tailrace excavation in view of diversion stages, future additional powerhouse units, and possibly a reregulating dam. Tests of the original structures were made with the tailrace excavated to elevation 2097 for full powerhouse operation without reregulation (plate 4). Subsequent tests were made with the channel at elevation 2110, which was designed for use with the initially installed powerhouse units or all units with reregulation. Plans included construction of powerhouse units 1 through 4 initially with units 5 through 8 to be added later; therefore, most model tests of the tailrace channel were made with partial powerhouse flows (10,000 cfs) released through units 1 and 2. The tests of the tailrace channel were made to develop optimum channel alignment compatible with diversion stage requirements and to determine the need for and design of bank revetment.

Plan A (Original Design)

28. Flow conditions with the Plan A channel (plate 28) with the maximum powerhouse flow of 44,000 cfs (8 units) were observed to study the channel alignment (plate 29). Eddies formed at the sharp corner on the right bank and in the pocket between the stilling basin and left bank. With a flow of 60,000 cfs (10,000 cfs powerhouse and 50,000 cfs Spillway), an eddy developed along the right bank and wave rideups of 9 to 16 feet occurred (plate 30). However, these were higher than expected in the prototype because of the smooth model banks.

Plan G (Recommended Plan)

29. The sharp angle on the right bank of the channel was replaced by a 570-foot-radius curve, and the left bank was aligned for second- and third-stage diversion (plate 31 and photograph 12). Flow conditions in the channel were studied and the stability of riprap along the banks was observed during sustained operation for 24 hours (prototype). Rocks having length- to minimum-width-ratios no greater than 3 and a uniform weight distribution of 750 to 1,500 pounds (W_{50}= 1,125 pounds) were placed, without effort to key them, on selected test areas to a depth of two rock diameters. The test areas were located where critical or typical wave impact and high velocities were noted. The bottom layer of rocks at the toe of slopes was grouted to the channel floor to simulate a buried toe (see typical section on plate 31). Other areas of the test banks were covered with 1,500- to 3,000-pound rocks (W_{50}=2,250 pounds) to the same thickness as the test areas.

30. Flow conditions in the channel were satisfactory with flows as high as 60,000 cfs (plates 32 to 34). With 10,000 cfs through the powerhouse and 50,000 cfs over the spillway, the maximum velocity near the bottom was 12 fps at the top of the stilling basin runout slope; a velocity of 9 fps existed just downstream from the runout slope (plate 34). The maximum wave height and rideup were 6 and 10 feet, respectively. When the spillway flow increased to 60,000 cfs, high-velocity surges from the stilling basin caused boiling over the runout slope, higher velocities around the runout area, and greater wave action (plate 35). Flow conditions were good with balanced conduit flow and the spillway closed. With unbalanced sluice flow, conditions were unsatisfactory.

31. With the design flow of 60,000 cfs, 750- and 1,500-pound riprap was satisfactory except in test area 3 (plate 31) near the powerhouse. The failure there was that of single rocks moving down the slope to a keyed position below the rideup zone. The high rideup did not move 1,500- to 3,000-pound rocks adjacent to the test area; therefore, that size riprap would be satisfactory in the area of

direct attack between stations 57+50 and 59+50 on the right bank. Minor failure occurred in test areas 1 and 2 on the right bank and 5 and 7 on the left bank when a greater-than-design flow of 70,000 cfs was passed through the structures (plate 35). The most damage occurred in area 5. The 750- to 1,500-pound riprap would have been adequate during normal operation; however, derrick stone (3,000 to 5,000 pounds) was to be placed on the left bank from the dam to test area 4 for protection during second and third-stage diversion.

Rock on Stilling Basin Runout Slope

32. Two sizes of rock were investigated for use on the 1V on 6H stilling basin runout slope: 3,000- to 5,000-pound derrick stone about 7 feet deep (two layers) and 1,500- to 3,000-pound rock 6 feet deep. Both sizes had approximately uniform graduation by weight and were dumped on the slope without keying individual rocks.

33. The 3,000 to 5,000-pound rock was required for the design flow of 50,000 cfs over the spillway and 10,000 cfs through the powerhouse. Only random, unkeyed rocks on the surface moved during 24 hours of operation (prototype); most of those rolled down the slope to the vicinity of the end sill. Violent boiling over the runout caused failure of the derrick stone protection when the basin design discharge was exceeded.

34. The 1,500- to 3,000-pound rock was not adequate. Although no damage occurred when 50,000 cfs was passed through the sluices, minor damage was noted when that discharge came over the spillway. Because the prototype rock will have a slightly lower specific gravity than that used in the model, use of the heavier rock was recommended.

Ejector Drains and Wall Pressures

35. Ejector drains in the floor and pressures on the left wall of the Plan D-1 stilling basin were studied to determine the effectiveness of the drains in relieving pressure buildup under the basin floor and to provide data for correlation with future pressure measurements

in the prototype.* Details of the basin and ejector drains and locations of the flush-mounted pressure cells are shown on plate 36. The five vertical rows of cells were at the same stations as proposed instrument slots to be installed in the right wall of the prototype.

36. Average pressures at the ejector drains are listed in table C for spillway discharges of 10,000 to 50,000 cfs. With a flow of 10,000 cfs, average drain pressures were 0 to 3 feet lower than average aerated water depths over the drains and with 20,000 to 40,000 cfs were 0 to 7 feet lower. At 50,000 cfs, the pressures were 3 to 6 feet less than average water depths at drains 1 to 9, 11 to 13 feet at drains 10 to 12, and 6 to 7 feet at drains 13 to 15. Part or all of the differences were due to heavy aeration of the water. The drains were relatively ineffective.

37. Instantaneous and mean pressures on the left wall are listed in table D. Within the design flow limits of the basin, the highest observed pressure was 76 feet at pressure cell 2 during a 50,000-cfs spillway discharge; the highest and lowest instantaneous pressures were 93 and -39 feet, respectively, at cell 5 with a sweepout flow of 100,000 cfs. The stilling basin was designed to pass a discharge of 100,000 cfs, an infrequent possible event, safely but not without the possibility of damage. Flows of 40,000 cfs occurred several times, and 50,000 cfs was approached during the initial months of project operation before power units were in use. The stilling basin was damaged by cavitation and debris abrasion. The entire floor and the lower 6 to 8 feet of the walls were eroded with the maximum erosion being in the center third of the floor. The top and the entire upstream sloping face of the end sill were severely damaged. The walls and floor adjacent to the sill were also heavily eroded. The lowest wall pressures with the discharges that have been experienced occurred in or immediately upstream from the area of maximum erosion.

* Hart, E. Dale, and Tool, Allen R., "Sluice Pressures, Gate Vibrations, and Stilling Basin Wall Pressures, Libby Dam, Kootenai River, Montana," Technical Report H-76-17, Oct 76, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Sluice Intakes

38. A combined study of the bellmouth intakes for the regulating sluices for the Libby and Dworshak projects* was made in a 1:2-scale model of the right sluices. The heads on the sluices and the discharge requirements for the two projects were similar. The outside sluices for both projects were skewed relative to the face of the structure to permit locating the intakes below the spillway abutments. An intake design was developed for Dworshak that was flush with the face of the dam and skewed to eliminate the need for corbels on the dam face. The Libby study was made to develop a similar design that also would be free of negative pressures. The Dworshak design could not be used because the 14.2-foot distance between the dam face and emergency gate slot at Libby was not adequate for the Dworshak design which required 17 feet. The plan designations are a continuation of those of the Dworshak study.

Plan F

39. The Plan F intake consisted of simple elliptical curves followed by converging tangents on the top and sides designed to establish discharge control at the upstream edge of the gate slot. Details are shown in photograph 3 and on plate 37. The design to shift flow control from the valve to the slot section was not standard practice but was an expediency for modification of plans of a project under construction. The study of the Dworshak design showed the shift to be instantaneous and stable with acceptable pressures. Pressures in the bellmouth, the gate slots, and the section between the slots and the valve are listed in table E. The piezometer locations are shown on plates 38 and 39. Flow conditions for free flow at pool elevation 2459 are shown in the top picture of photograph 13. The minimum pressure in the bellmouth was -12 feet at piezometer 16F,

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* "Dworshak Dam, North Fork Clearwater River, Idaho," Technical Report No. 116-1, U.S. Army Engineer Division, North Pacific, Division Hydraulic Laboratory, CE, Bonneville, OR, to be published in 1983.

which indicated possible cavitation. Negative pressures existed along the downstream one-half of the left bottom corner. Pressures in the other corners were positive. On the right side, flow impinged intermittently on the downstream edge of the slot near the top of the sluice, and on the left it continually impinged along one-fourth to one-half of the downstream edge of the slot. The lack of impact pressures at piezometers 33F, 34F, 36F, and 37F (located 0.2 foot from the edges) indicated that the impingement was limited to the edges of the slots. No adverse pressures at piezometers 38F, 45F, and 52F developed from separation of flow at the edges of the slots. With the flow uncontrolled, free surface flow occurred downstream from the slots when a small amount of air was admitted through the slots.

Plan G

40. To increase the throttling effect and thus create additional back pressure from the control area at the upstream edge of the emergency gate slots, the roof convergence was increased from 8.5 to 14.0 degrees (plate 40). The bottom and sides were not changed. The 3-5/8-inch setback at the downstream edge of the slots was increased to 6 inches. This was the maximum possible increase without redesigning the slots and gate.

41. Pressures at the piezometer locations shown on plates 41 and 42 are listed in table F. Flow conditions for free flow at pool elevation 2459 are shown in the bottom picture of photograph 13. The minimum pressure--4 feet at peizometer 11G--satisfied the criterion for design. Although the setback at the downstream edges of the slots was increased, the increased roof convergence caused greater impingement on the edges of the slots. As in the previous design, pressures adjacent to the slots were satisfactory. The impact area and about 40 feet of the prototype floor and walls downstream from the slots are protected with a steel liner.

42. When the slot was sealed and air excluded (a condition that would not occur in the prototype), control of flow shifted to the valve section, full sluice flow occurred upstream from that point, and

minimum pressures were -12 feet in the bellmouth and -25 feet in the slots (table G). The amount of air required through the slot was not measured.

43. The discharge rating of the sluice with fully open valve is shown on plate 43. The maximum discharge 19,990 cfs at pool elevation 2459 was 340 cfs less than computed. The maximum capacity of three sluices would be 59,970 cfs instead of 61,000 cfs. At minimum pool elevation 2287, the maximum capacity was 10,930 cfs, 740 cfs less than anticipated.

44. After the model tests were completed, the decision was made to construct corbels on the face of the dam that would contain intakes perpendicular to the sluice alignments. The intake design was similar to Plan D selected for Dworshak (plate 44) prior to the development of the adopted Plan E having skewed intakes.

45. After 18 months of operation, cavitation was discovered on the invert and walls of the center sluice downstream from the valve and on the roof immediately upstream from the valve; ten months later cavitation damage was found in the right sluice. Repairs were made, and the center sluice was field tested to determine the cause of the damage.* Model tests were made by the U.S. Army Engineer Waterways Experiment Station to develop an aerator as a means to prevent future cavitation damage.**

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* See footnote page 16.

** Dortch, Mark S., "Center Sluice Investigation, Libby Dam, Kootenai River, Montana," Technical Report H-76-21, Dec 76, U.S. Army Waterways Experiment Station, CE, Vicksburg, MS.

PART III: TESTS OF DIVERSION STAGES

Second and Third Diversion Stages

Proposed Construction Schedule

46. Portions of the Plan G channel, a diversion channel and first-stage cofferdam, right bank Monoliths 1 to 8, portions of Monoliths 28 to 37, and the stilling basin were to be installed during the first stage of construction. Monolith locations are shown on plate 1. Monoliths 33 to 35 would be left low for second-stage diversion and temporary sluices provided in Monoliths 32, 34, and 36 (plate 45). After the 1968 flood season the first-stage cofferdam would be breached and the river allowed to flow over diversion Monoliths 33 to 35. Monolith 33 was to be raised after high water in the summer of 1970. The temporary sluices were to be opened in the spring or early summer of 1971.

47. The third stage of construction, from August 1971 to about October 1972, was to start with raising diversion Monolith 35 and passing all flow through the temporary sluices in Monoliths 32, 34, and 36. As the monoliths were raised, the temporary sluices would be closed one at a time with an intake gate. Flow through the last temporary sluice would be regulated to a minimum of 2,000 cfs while the reservoir filled until this minimum discharge could be passed through the permanent regulating sluices.

Plan D Diversion (Original Design)

48. Model studies of the first construction stage were not required. Details of the Plan A (original contract plan) first and second steps of second-stage diversion are shown on plates 45 and 46. Third-stage diversion is shown on plate 47. The left bank and the backfill at the toe of the dam were to be protected by 3,000- to 5,000-pound derrick stone. Model tests indicated that the plan was not satisfactory during second-step, second- and third-stage diversion. High velocities caused extensive damage to model riprap at the toe of the dam and the adjacent left bank. Since that portion of the

left bank would support the roadbed for a transcontinental railway, no damage to the bank was permissible. The high velocities of flow through the diversion structure dissipated slowly, and extensive erosion of the relatively shallow channel was indicated.

Recommended Plan

49. Details of the recommended plan (Plan F diversion) are shown in photographs 14 and 15 and on plate 48. The structural changes included: (1) replacement of the sluice in Monolith 36, which spilled onto the left bank, with one in Monolith 31, (2) provision for stop-logs near the downstream end of Monolith 33 to cause the outflow to plunge, spread, and dissipate energy better, and (3) addition of a low wall along the left bank adjacent to the dam. Excavation around the structures which was riprapped and backfilled in Plan A was left open. The derrick stone on the left bank above the toe wall was grouted, and the derrick stone and riprap on the remainder of the downstream bank slopes were loose-dumped.

50. Diversion was studied with a continous test from first-step, second-stage diversion to third-stage diversion. Velocities over the riprapped banks were measured in flows of 60,000, 82,000, and 109,000 cfs with first- and second-step, second-stage diversion and 20,000 and 30,000 cfs with third-stage diversion. The flow was interrupted while scour and deposition patterns were observed after each discharge. The gradation of the movable-bed material used in the tests is shown on plate 49.

51. Velocities on the banks during first-step, second-stage diversion are shown on plates 50 to 52. Flow conditions are shown in photographs 16 and 17. Almost no movement of the initial channel bed occurred with the flow of 20,000 cfs. With 30,000 to 50,000 cfs, bed movement was slight and stablizied in less than 4 hours. With 60,000 cfs, the scour did not stabilize in 10 hours and individual derrick stones on the left bank were dislodged. Just downstream from the two lowest monoliths scour depth extended to bedrock. After the 82,000-cfs flow, scour along the left bank exposed the toe of the derrick stone

downstream from the wall; however, no failure of the bank protection occurred. At 100,000 and 109,000 cfs the derrick stone withstood the flow, but scouring occurred on the stilling basin runout slope downstream from the derrick stone (photograph 17).

52. During second-stage diversion (photographs 18 and 19), the higher concentration of flow from Monolith 33 moved the scour pattern downstream and to the right. With flows to 82,000 cfs, a small amount of fine material was deposited in the stilling basin (photograph 20). Deposition occurred in the prototype stilling basin during diversion. Most, but not all, of the debris was removed prior to use of the basin. The debris that was left apparently contributed to the subsequent erosion of the basin. The deposition in the stilling basin ceased at 109,000 cfs, and derrick stone was washed away near the corner of the stilling basin wall (photograph 21). Since the basin wall was to be founded on solid rock, this erosion would not endanger the structure. The maximum velocity over ungrouted derrick stone on the banks was 15 fps (plates 53 to 55).

53. Bank velocities were low (plates 56 and 57), and only minor erosion occurred during the low flows of third-stage diversion (photographs 22 and 23). Almost all movement was along the right side. Material deposited downstream from Monolith 31 was moved into the scoured area downstream from the stilling basin by flows of 10,000 to 20,000 cfs and on downstream by 30,000 cfs (photograph 23).

Closure of Monolith 33

54. Water surface elevations were measured as Monolith 33 was closed at the end of second-stage diversion by means of a 61-foot-wide closure bulkhead on the upstream face of the dam. Details of the diversion structure are shown on plate 48. With a river discharge of 20,000 cfs, the data in table H was obtained to determine the backwater in the low monolith and the head on the closure gate with 10.4-foot-high stoplogs near the downstream end of the monolith. With steady flow, the forebay elevation varied from 2137.0 when the gate was fully open to 2146.8 when the gate was lowered to an opening of

5.0 feet. Average water surface elevations at the center of the monolith decreased from 2135.0 to 2131.5. Heads on the gate varied from 6.2 feet with a gate opening of 15.0 feet to 21.8 feet with an opening of 5.0 feet.

Debris Deflectors for Construction Trestle

55. As shown on plate 48, Monoliths 33 and 35 would be completed to elevation 2110 and Monolith 34 to elevation 2140 during first-stage construction. Flow would be passed over these monoliths during the first step of second-stage diversion (paragraph 46). Construction methods included a temporary steel trestle tower on Monolith 34 (plate 58). The tower legs would be subjected to the impact of water flowing over Monolith 34 during river discharges greater than about 50,000 cfs. Debris might collect on the legs, reduce flow over the monoliths, and raise the upstream water surface enough to endanger the railway roadbed at elevation 2160. This was especially likely during the spring of 1969 when there was a possibility of a flood greater than the 109,000 cfs for which the diversion facilities were designed.

56. Two deflectors that diverted all flow around the tower were tested in the model. One had a blunt (180-degree) upstream face; the other had a 90-degree nose (plate 59). Both created backwater effects that raised the forebay 1.3 and 1.1 feet, respectively, at a flow of 109,000 cfs. The maximum wave rideups on the two deflectors were 17.5 and 16.5 feet, respectively. A 40-foot-long triangular float anchored 50 feet upstream from the dam on the centerline of Monolith 34 deflected all debris around the tower and had no effect on pool levels. The device was abandoned because the float would be difficult to anchor in the gravel bed upstream.

57. The adopted plan included two 15-foot-high concrete piers that encased the tower legs (photograph 24 and plate 60). Each pier served as a deflector and would become part of the mass concrete of the monolith. Debris that caught on the long pier noses could be dislodged or removed by means of overhead cranes. Flow around the piers was satisfactory. Debris caught on a pier nose and on a cross

member of the model tower is shown in photograph 24. The piers had no significant effect on the forebay elevation during a riverflow of 109,000 cfs. Maximum rideup of waves on the piers was 14 feet.

PART IV: TESTS AFTER INITIAL PROTOTYPE OPERATION

Flow Required to Clean Stilling Basin

58. After a period of initial operation, silt and gravel were found in the prototype stilling basin. Previous model tests had indicated that a deposition might occur in the downstream left corner. Tests were made to determine the sluice flow required to wash this debris out of the basin. Coarse sand and 3/8-inch-minus gravel that simulated gravel and rocks weighing up to 200 pounds were deposited against the end sill. Total sluice flows of 5,000, 10,000, 18,000, and 25,000 cfs were passed in succession for a 24-hour period (3 hours and 24 minutes in the model) at each discharge. The two lower flows had little effect on the deposited material. At 18,000 cfs, some of the material was swept out of the basin and deposited downstream from the end sill and behind the left basin wall. All the remaining debris was swept from the basin by a flow of 25,000 cfs. Some of the material was deposited just downstream from the end sill; the rest settled in the exit channel along the edges of the outflow jet.

59. Silt and gravel were swept from the prototype stilling basin; however, the completeness of cleaning was not determined at the time and within a year material was found in the basin again. Subsequent examination of the basin revealed erosion damage to the floor and end sill; the damage has been repaired.

Reduction of Absorbed Nitrogen

60. Flow through high-head spillways and hydraulic-jump stilling basins aerates heavily. In deep stilling basins, large quantites of the air may be absorbed and retained as dissolved gases of which nitrogen is predominent. Since all flow was to be released through the stilling basin until power units were in operation, model studies were made in an attempt to develop modifications that would reduce nitrogen absorption with discharges to 40,000 cfs. The studies discussed in Appendix A did not lead to development of a satisfactory modification.

TABLE A

PRESSURES ON PLAN A STRUCTURES

Spillway Discharge 145,000 CFS, Free Flow, Sluices Closed

Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water
Abutment		Crest		Sluice Exits	
A- 1	38	C- 1	26	E- 1	0
A- 2	22	C- 3	6	E- 2	0
A- 3	26	C- 4	-4		
A- 4	2	C- 5	-4	E- 3	6
A- 5	0			E- 4	10
		C- 6	3	E- 5	14
A- 6	0	C- 7	2	E- 6	51
A- 7	4	C- 8	1		
A- 8	-6	C- 9	3	E- 7	14
A- 9	-1			E- 8	52
A-10	-6	C-10	6		
		C-11	12		
A-11	-3	C-12	9	E- 9	0
A-12	-5	C-13	9	E-10	0
A-13	0				
A-14	1			E-11	3
A-15	-1	C-14	20	E-12	8
		C-15	14	E-13	13
A-16	0	C-16	6	E-14	45
A-17	-1	C-17	4		
A-18	0	C-18	1		
A-19	3				
A-20	9	C-19	-2		
		C-20	-3		
		C-21	0		
Pier		C-22	5		
		C-23	11		
P- 1	47				
P- 2	2[illegible]	C-24	9		
P- 4	3	C-25	7		
P- 5	5	C-26	9		
P- 6	1	C-27	9		
P- 7	-9				
P- 8	-12	C-28	28		
P- 9	-2	C-29	8		
P-10	-9	C-31	-2		
P-11	-2	C-32	-3		
P-12	-6	C-33	3		
P-13	1	C-34	-1		
P-14	2	C-35	-7		
P-15	-1	C-36	6		
P-16	-1				
		C-37	5		
P-17	-5	C-38	12		
P-18	-6	C-39	10		
P-19	3	C-40	8		
P-20	6				
P-21	4				
P-22	9				
P-23	9				

NOTE: Piezometer locations are shown on plate 10.

TABLE B

PRESSURES ON PLAN B SPILLWAY

(Plan A Crest, Plan B Pier and Abutments)

Pressures on Crest		
Spillway Flow in CFS	60,000	145,000
Gate Opening in Feet	18.0	Free Flow
Piezometer Number	Pressure in Feet of Water	
C-14	50	23
C-15	46	18
C-16	40	10
C-17	35	7
C-18	16	3
C-19	1	0
C-20	-3	-2
C-21	-1	-1
C-22	2	4
C-23	5	10
C-24	5	8
C-25	4	7
C-26	4	8
C-27	3	5
C-28	53	34
C-29	45	17
C-31	35	-2
C-32	33	1
C-33	16	3
C-34	1	1
C-35	-3	-2
C-36	1	-1
C-37	3	6
C-38	4	6
C-40	4	7

Pressures on Abutment - Free Flow		
Spillway Flow in CFS	50,000	145,000
Head on Crest in Feet	27.8	52.9
Piezometer Number	Pressure in Feet of Water	
A-21	19	19
A-22	15	9
A-23	10	-2
A-24	7	-11
A-25	5	-15
A-26	4	-16
A-27	4	-15

NOTES:
1. Details of crest are shown on plate 8.
2. Details of pier and abutments are shown on plate 16.
3. Piezometer locations are shown on plate 18.
4. No piezometers in pier.

TABLE C

EJECTOR DRAIN PRESSURES

Plan D-1 Stilling Basin

Drain No.	Spillway Discharge in CFS				
	10,000	20,000	30,000	40,000	50,000
	Average Pressure at Drain Opening - Feet of Water				
1	51	44	39	37	33
2	50	44	39	37	31
3	51	45	41	39	34
4	51	50	47	43	39
5	51	50	45	44	39
6	52	51	47	47	41
7	52	52	52	52	49
8	52	53	52	53	50
9	52	53	53	54	50
10	53	54	55	55	55
11	53	55	54	54	54
12	53	55	55	55	56
13	53	55	55	57	57
14	53	55	56	56	57
15	53	55	56	58	58

NOTES: 1. Details of stilling basin and location of ejector drains are shown on plate 36.

2. River flow controlling tailwater elevations equalled spillway discharge plus 10,000 cfs through powerhouse.

TABLE D

WALL PRESSURES

Plan D-1 Stilling Basin

Pressure Cell No.	Spillway Discharge in CFS																				
	10,000			20,000			30,000			40,000			50,000			100,000			145,000		
	Pressure in Feet of Water																				
	Instantaneous		Mean	Instantaneous		Mean	Instantaneous		Mean	Instantaneous		Mean	Instantaneous		Mean	Instantaneous		Mean	Instantaneous		Mean
	Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.		Max.	Min.	
1	42	21	28	41	2	23	60	-6	17	65	-29	18	65	-19	10	23	-	-	10	-	-
2	46	31	37	49	-3	33	58	-4	29	71	-5	28	76	-22	26	14	-	-	10	-	-
3	42	33	38	50	21	38	60	13	37	62	7	38	64	-3	33	26	-	-	8	-	-
4	43	40	41	47	35	43	54	27	43	58	23	44	67	21	44	78	-13	5	52	-	-
5	39	39	39	43	39	41	46	35	42	50	32	42	56	27	45	93	-39	22	66	13	50
6	18	11	14	16	1	8	11	-4	4	13	-2	1	7	-	-	-	-	-	-	-	-
8	17	12	16	23	7	16	30	2	16	36	0	14	31	-8	10	7	-	-	-	-	-
9	15	9	13	20	10	14	26	7	16	26	0	16	31	-3	16	60	-9	0	-	-	-
10	19	17	18	24	15	20	27	15	22	31	1	23	33	9	24	70	-18	9	-	-	-
11	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
12	1	-	-	2	-	-	3	-	-	3	-	-	5	-	-	-	-	-	-	-	-
13	3	-	-	6	-	-	11	-	-	7	-	-	13	-	-	-	-	-	-	-	-
14	-	-	-	7	-	-	7	-	2	13	-	3	13	-4	2	-	-	-	-	-	-
15	-	-	-	3	-	-	7	-	1	10	-	2	13	-4	5	-	-	-	-	-	-

- Dry

NOTES:
1. Pressures measured with flush mounted, 1/2-in.-dia, 25-psia pressure cells.
2. Pressure cell locations and stilling basin details shown on plate 36. Pressure cell No. 7 not operable.
3. River flow controlling tailwater elevations equalled spillway discharge plus 10,000 cfs through powerhouse.

TABLE E

MINIMUM PRESSURES WITH PLAN F BELLMOUTH IN RIGHT SLUICE

Pool Elev 2459, Fully Open Valve, Discharge 20,410 CFS
Emergency Gate Slot Open

Pressures in Bellmouth				Pressures in Section Between Bellmouth and Valve			
Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water
1	43	17	160	32	*	48	3
2	31	18	108	33	*	49	6
3	21	19	81	34	*	50	6
4	58	20	45	35	3	51	7
5	33	21	191	36	7	52	*
6	27	22	133	37	*	53	54
7	80	23	90	38	*	54	-1
8	17	24	66	39	*	55	1
9	-8	25	153	40	4	56	2
10	-3	26	115	41	8	57	2
11	-8	27	86	42	10	58	2
12	37	28	27	43	14		
13	0	29	129	44	15		
14	1	30	103	45	104		
15	-4	31	67	46	35		
16	-12			47	3		

* Aerated flow; pressures approximately atmospheric

NOTES:
1. Details of bellmouth are shown on plate 37.
2. Piezometer locations are shown on plate 38 and 39.
3. Pressures were measured by means of water manometers.

TABLE F

MINIMUM PRESSURES WITH PLAN G BELLMOUTH IN RIGHT SLUICE

Pool Elev 24[illegible] Fully Open Valve, Discharge 19,990 CFS
Emergency Gate Slot Open

Pressures in Bellmouth				Pressures in Section Between Bellmouth and Valve			
Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water
1	52	17	169	32	14	48	8
2	39	18	134	33	11 to 12	49	4
3	19	19	112	34	1 to 5	50	9
4	69	20	62	35	7	51	9
5	45	21	195	36	5 to 6	52	*
6	28	22	147	37	*	53	49 to 74
7	92	23	125	38	14	54	17
8	30	24	94	39	21	55	2
9	11	25	169	40	24	56	-1
10	10	26	142	41	16	57	2
11	4	27	110	42	15	58	2
12	52	28	27	43	18		
13	12	29	152	44	17		
14	15	30	127	45	61 to 80		
15	8	31	83	46	75		
16	6			47	31		

* Aerated flow; pressures approximately atmospheric

NOTES:
1. Details of bellmouth are shown on plate 40.
2. Piezometer locations are shown on plates 41 and 42.
3. Pressures were measured by means of water manometers.

TABLE G

MINIMUM PRESSURES WITH PLAN G BELLMOUTH IN RIGHT SLUICE

Pool Elev 2459, Fully Open Valve, Discharge 20,710 CFS
Emergency Gate Slot Closed

Pressures in Bellmouth				Pressures in Section Between Bellmouth and Valve			
Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water	Piezometer Number	Pressure in Feet of Water
1	38	17	165	32	-4	48	3
2	23	18	125	33	-14	49	-1
3	4	19	103	34	-15	50	5
4	56	20	44	35	-11	51	6
5	31	21	193	36	-21	52	46
6	13	22	142	37	-25	53	41
7	79	23	114	38	21	54	17
8	13	24	82	39	35	55	1
9	-5	25	164	40	25	56	-2
10	-4	26	136	41	12	57	1
11	-12	27	97	42	12	58	2
12	37	28	0	43	15		
13	-4	29	144	44	15		
14	-1	30	118	45	94		
15	-7	31	67	46	60		
16	-10			47	23		

NOTES: 1. Details of bellmouth are shown on plate 40.

2. Piezometer locations are shown on plates 41 and 42.

3. Pressures were measured by means of water manometers.

TABLE H

WATER-SURFACE ELEVATIONS

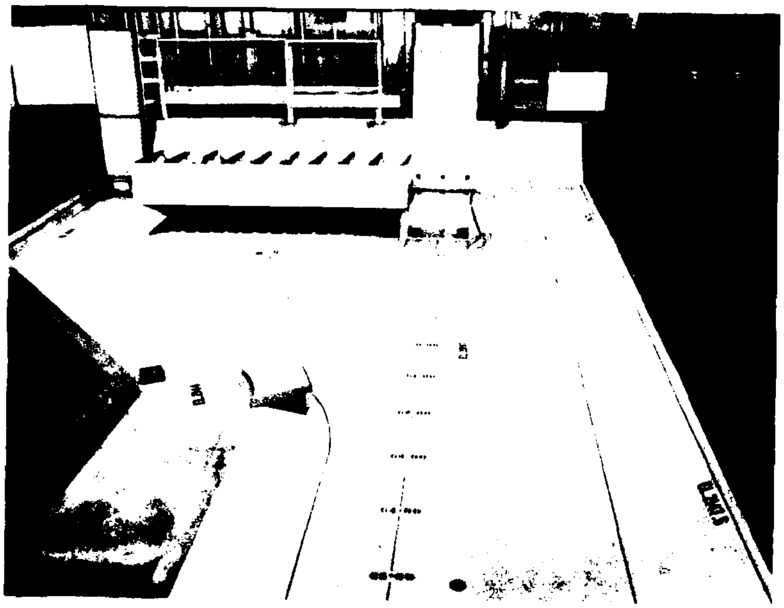
Closure of Low Monolith 33
Plan F Second- and Third-Stage Diversion
River Discharge 20,000 CFS, Tailwater Elev 2125.1

Closure Gate Opening in Feet	Water-Surface Elevation in Feet-MSL						
	Forebay	Downstream Face of Gate			Center of Monolith		
		Maximum	Minimum	Average	Maximum	Minimum	Average
Fully Open	2137.0						2135.0
15.0	2138.2	2132.5	2131.0	2132.0	2135.0	2133.5	2134.5
10.0	2140.3	2131.0	2128.5	2129.0	2136.0	2132.5	2134.5
5.0	2146.8	2126.5	2123.0	2125.0	2133.0	2129.5	2131.5

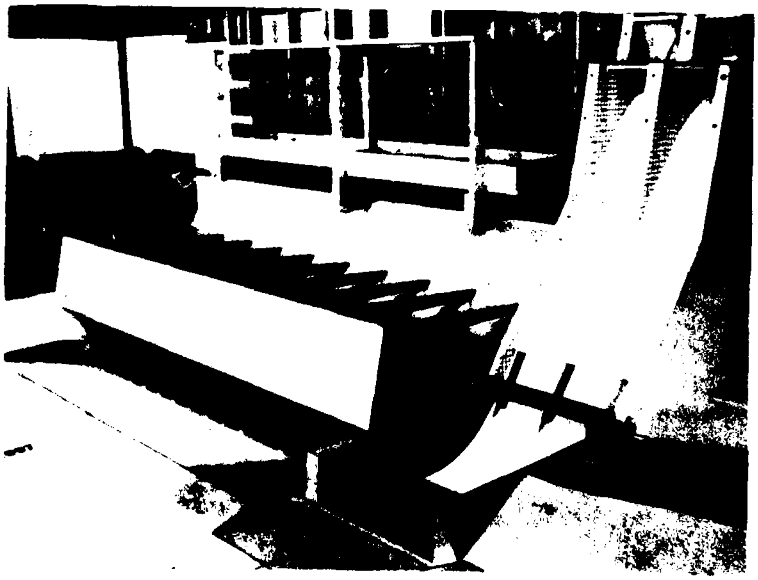
NOTES: 1. Details of diversion structures are shown on plate 48.

2. Closure gate on upstream face of monolith.

3. Stop logs near downstream end of monolith 10.4 ft high.



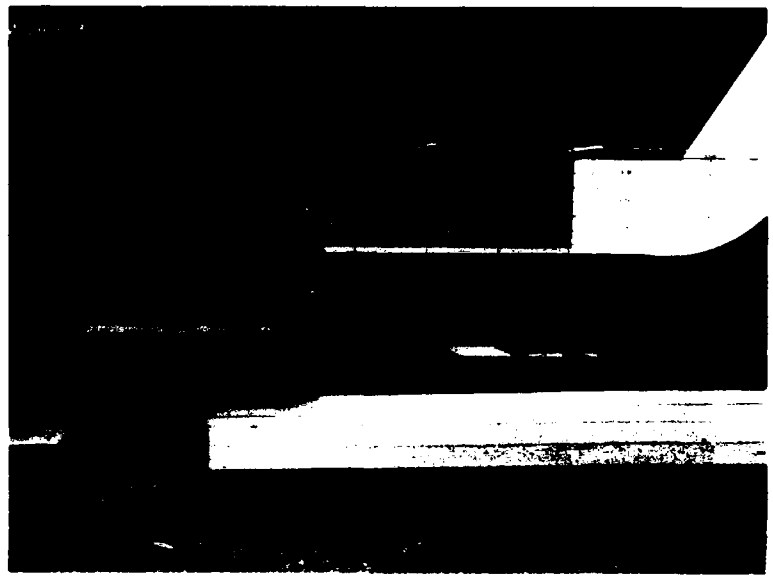
View looking upstream



Spillway and powerhouse

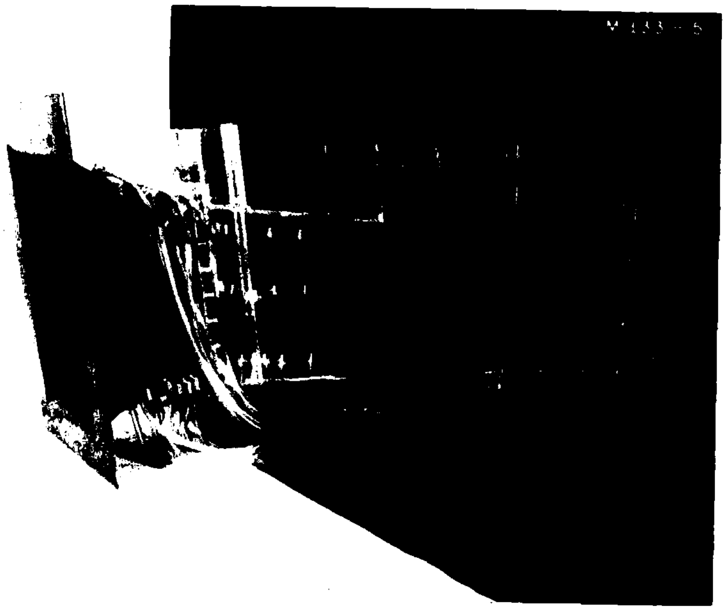
Photograph 1. Plan A (original design) structures and tailrace in 1:50-scale spillway model.

View looking upstream

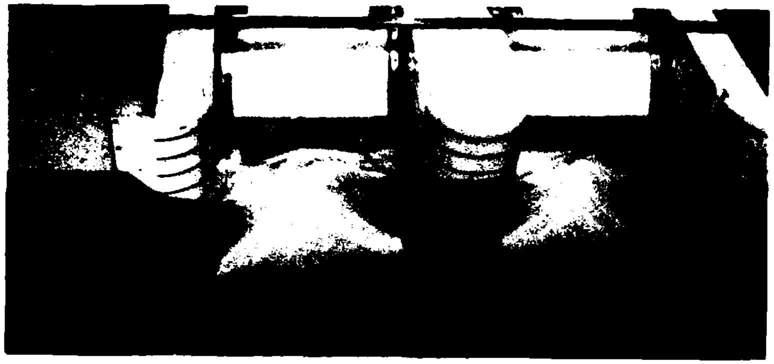


Side view of stilling basin, runout slope, and excavated tailrace

Photograph 2. The 1:50-scale spillway model used for nitrogen reduction studies.

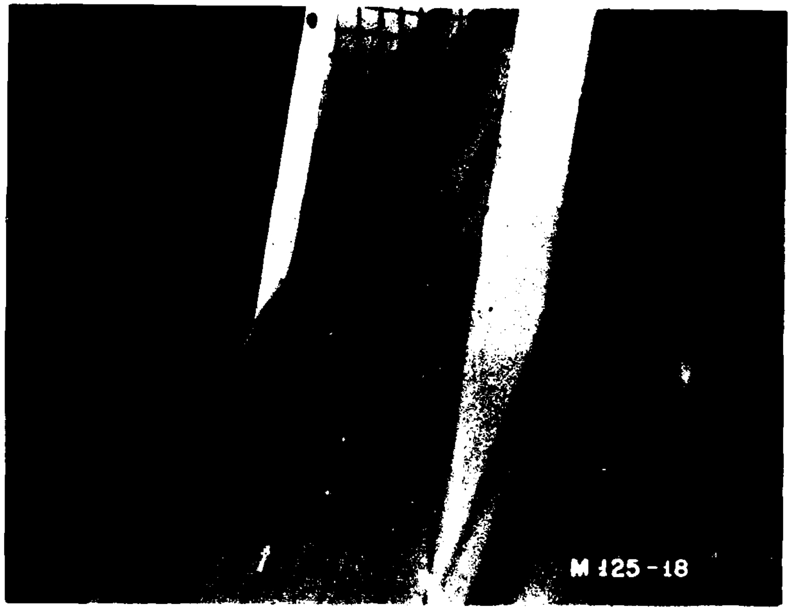


Photograph 3. Plan F bellmouth and valve section for 1:20-scale model of regulating sluice intake.



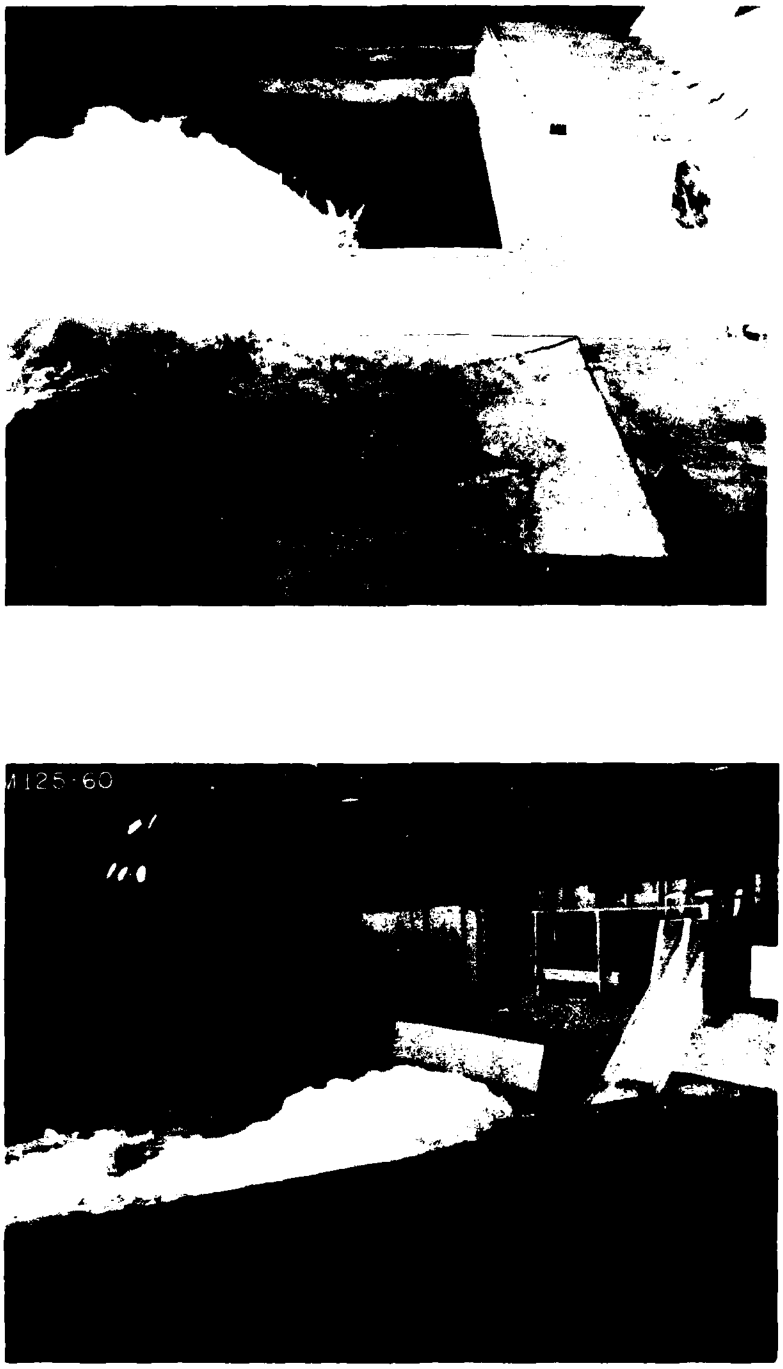
Inflow to spillway (bridges removed)

Overtopping of chute wall at sluice exits
and impingement of rooster tail on bridge



Overtopping of chute wall at abutment
Top of prototype wall indicated by black line

Photograph 4. Flow conditions at plan A (original design) structures; spillway discharge 145,000 cfs.



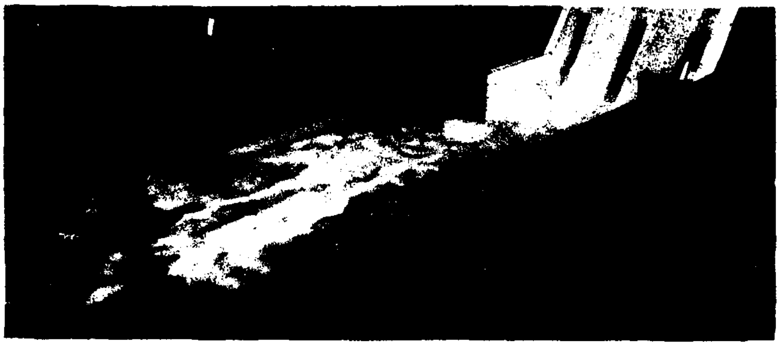
Photograph 5. Flow conditions with plan B stilling basin. Spillway discharge 145,000 cfs, sluice discharge 61,000 cfs, tailwater elev 2144.0.

Spillway discharge 145,000 cfs, free flow; no overtopping of pier extension

Spillway discharge 110,000 cfs, gate opening 32.8 ft, occasional overtopping of pier extension

Photograph 6. Flow conditions with plan B pier and abutments and plan A-1 pier extension.

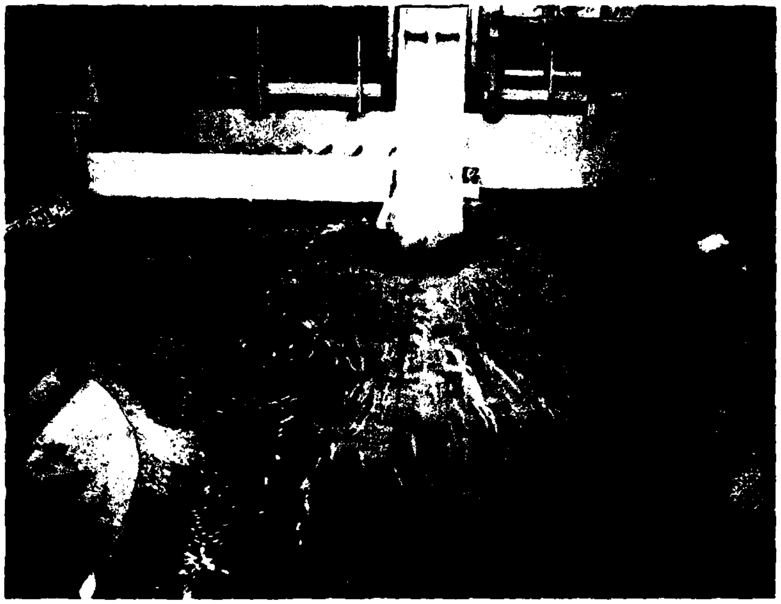
Spillway discharge 50,000 cfs, powerhouse discharge 10,000 cfs
tailwater elev 2131.7



Spillway discharge 51,000 cfs, powerhouse closed
tailwater elev 2130.2

Sluice discharge 50,000 cfs, powerhouse discharge 10,000 cfs
tailwater elev 2131.7

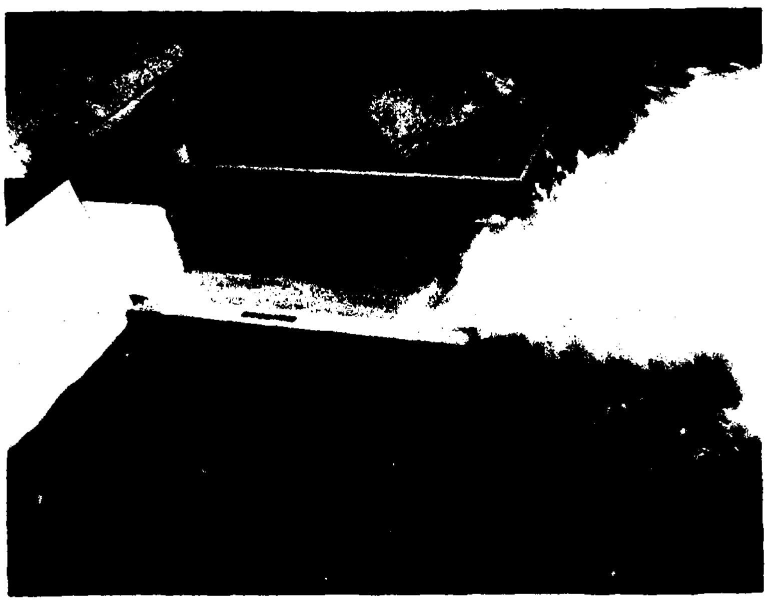
Photograph 7. Flow conditions with plan D stilling basin and plan A interim channel.



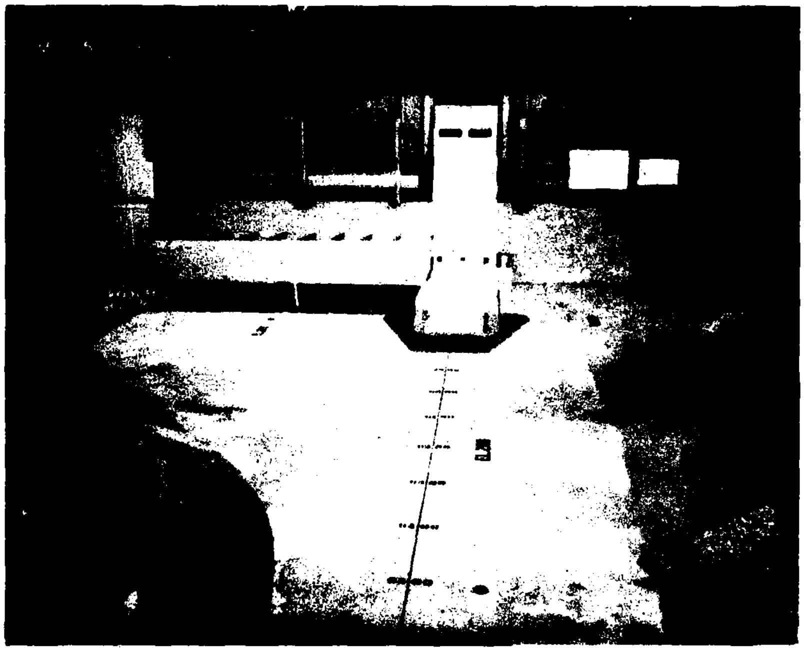
Photograph 8. Flow conditions with plan D-1 stilling basin and plan G downstream channel. Spillway discharge 50,000 cfs, powerhouse discharge 10,000 cfs, tailwater elev 2131.7.

Photograph 9. Flow conditions with plan D-1 stilling basin and plan G downstream channel. Spillway discharge 60,000 cfs, powerhouse discharge 10,000 cfs, tailwater elev 2133.1.

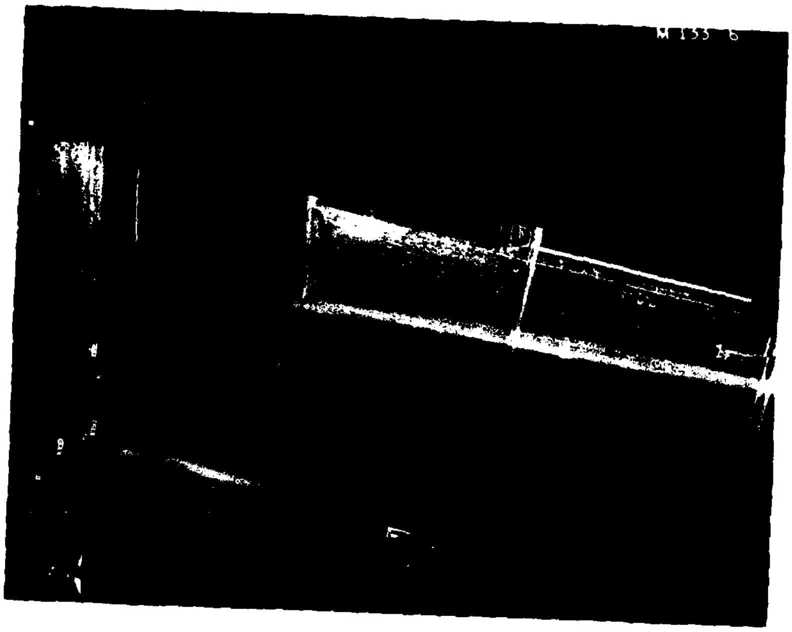
Photograph 10. Flow conditions in plan D-1 stilling basin and on runout slope. Spillway discharge 100,000 cfs, tailwater elev 2138.1.



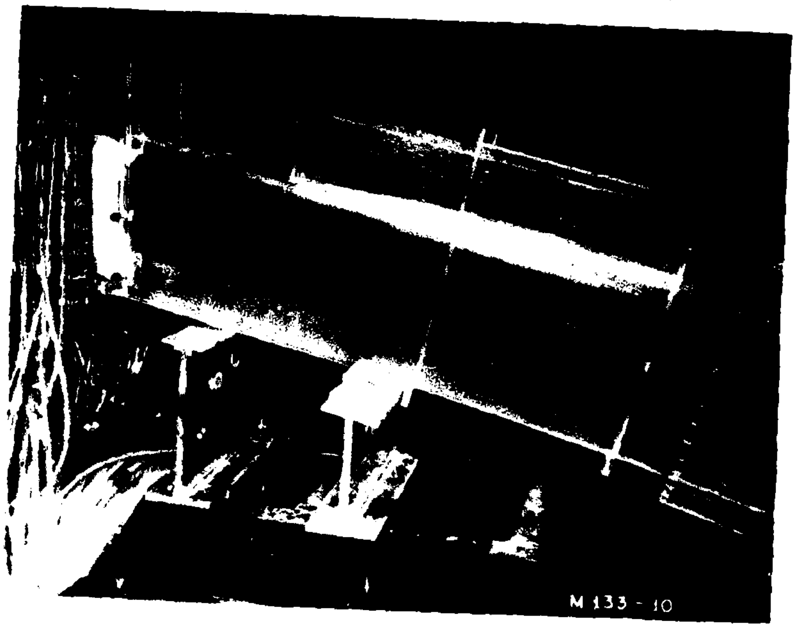
Photograph 11. Flow conditions in plan D-1 stilling basin and on runout slope. Spillway discharge 145,000 cfs, tailwater elev 2143.0.



Photograph 12. Plan G downstream channel with plan D-1 stilling basin.



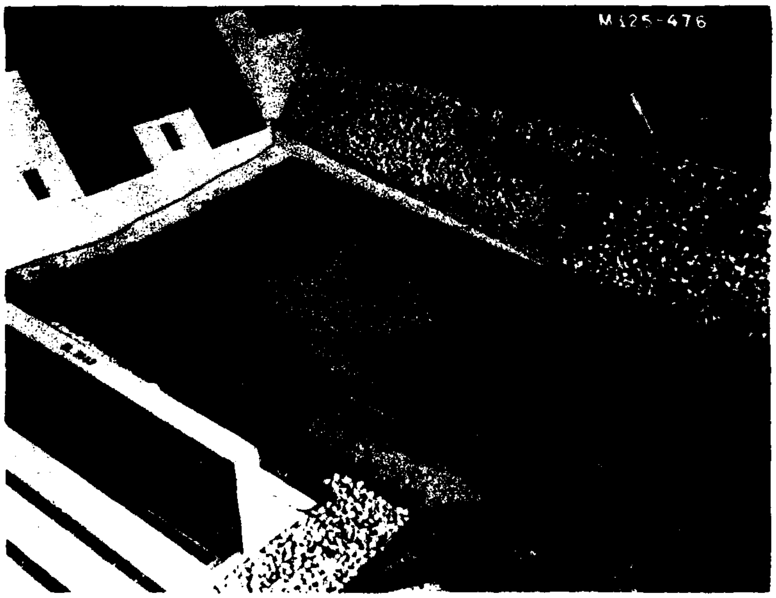
Plan F
Flow separation caused by impingement in right slot



Plan G

Photograph 13. Flow conditions with bellmouth plans F and G. Free flow, pool elev 2459.0.

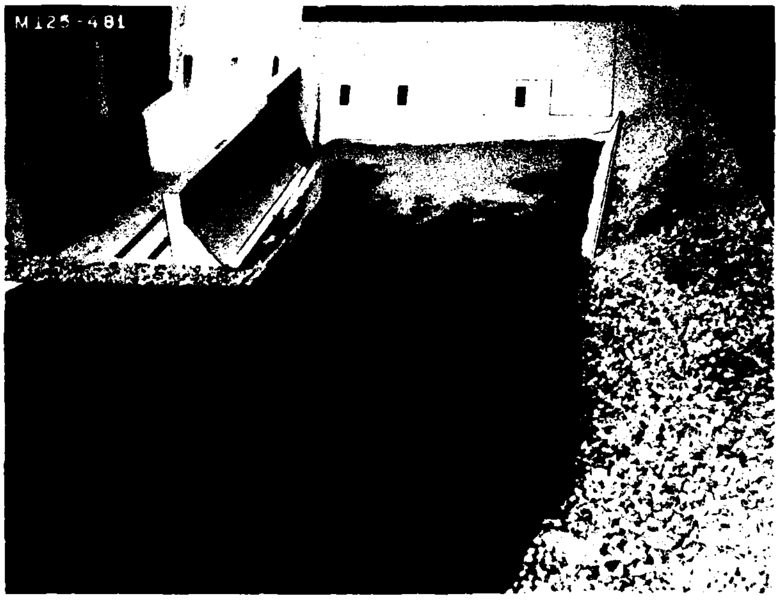
View from downstream



Derrick stone on stilling basin runout
slope and riprap along left bank

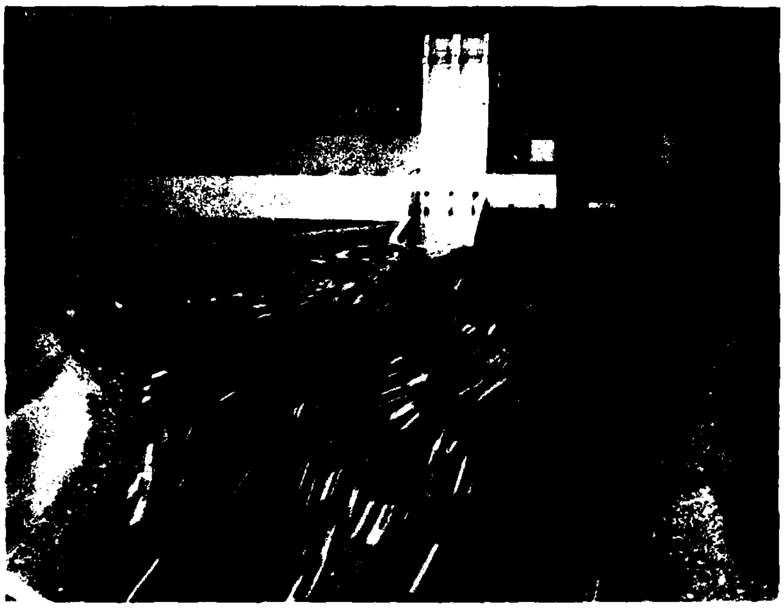
Photograph 14. First-step, second-stage diversion structures and downstream channel (before scour tests).

Second-step, second-stage diversion



Third-stage diversion

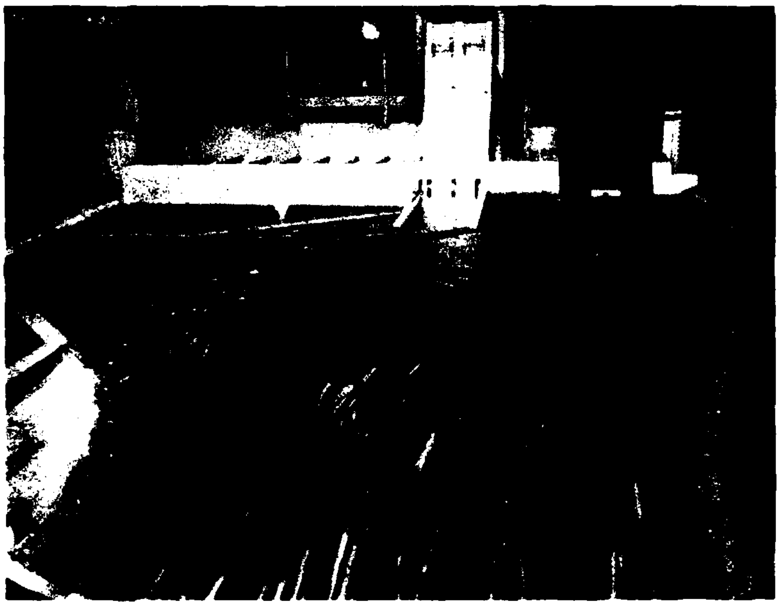
Photograph 15. Diversion structures and downstream channel (before scour tests).



Discharge 60,000 cfs

Discharge 82,000 cfs

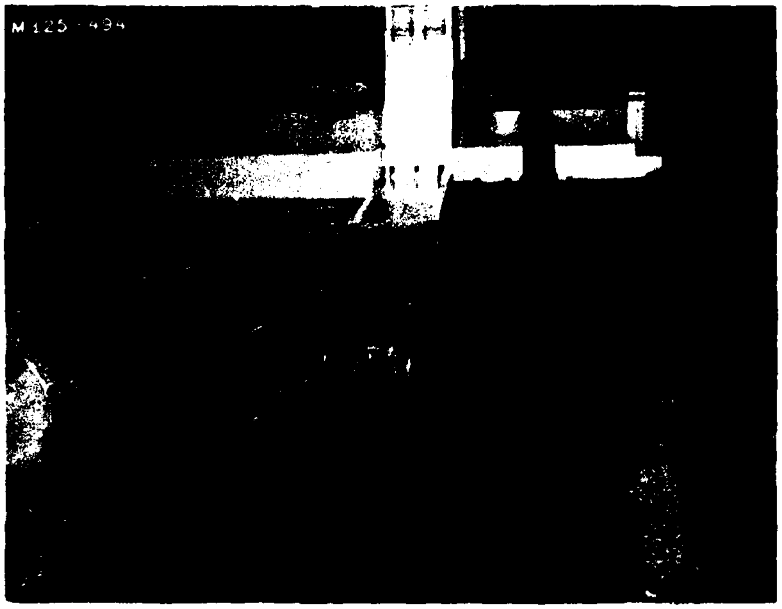
Photograph 16. Flow conditions during first-step, second-stage diversion.



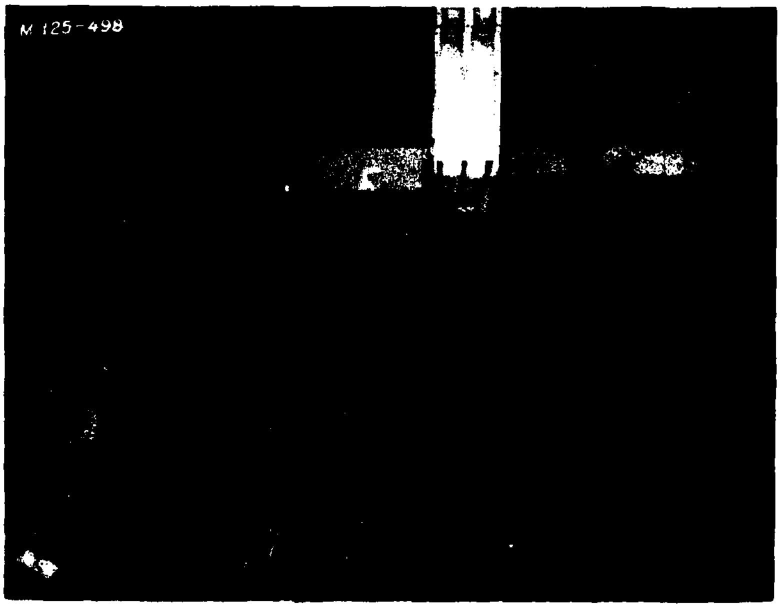
Flow conditions

Scour and deposition after 10 hours

Photograph 17. First-step, second-stage diversion.
Discharge 109,000 cfs.

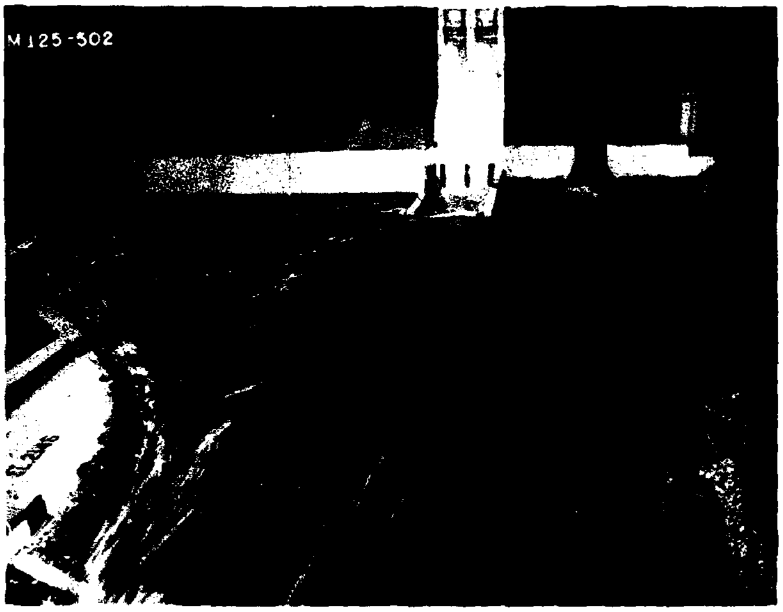


Discharge 60,000 cfs

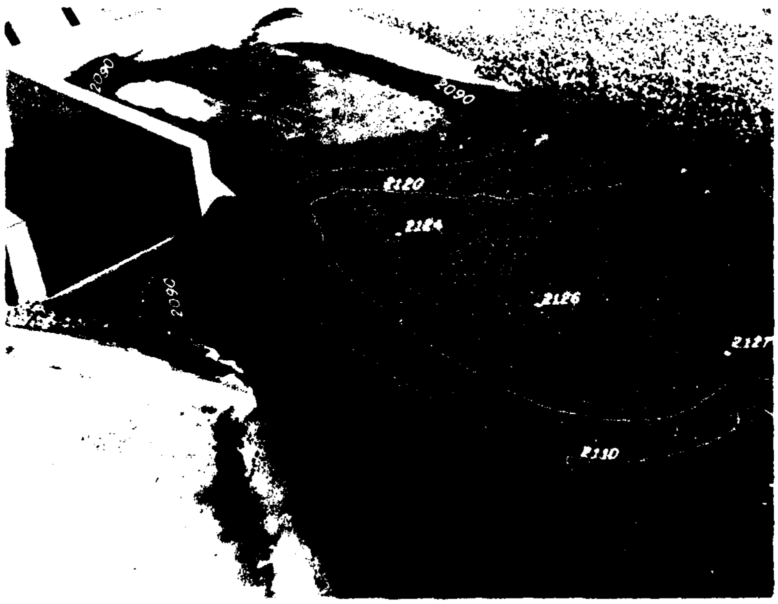


Discharge 82,000 cfs

Photograph 18. Flow conditions during second-step, second-stage diversion.

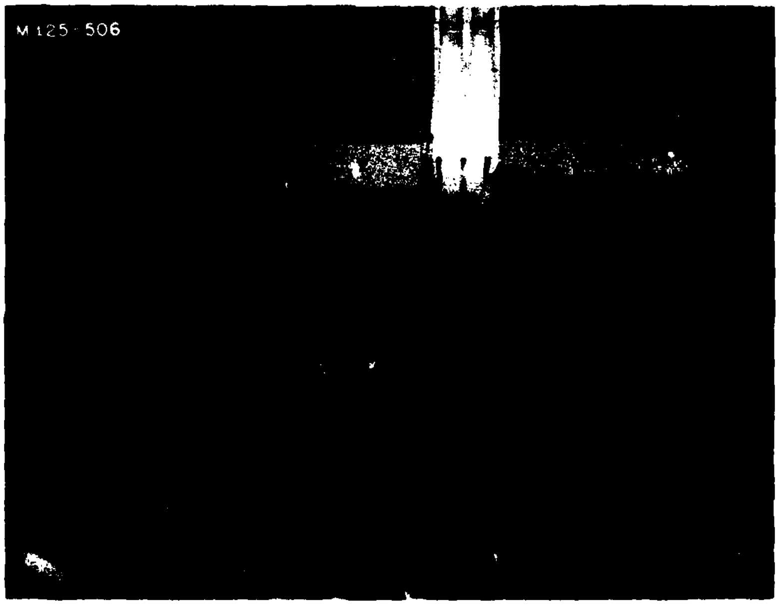


Photograph 19. Flow conditions during second-step, second-stage diversion. Discharge 109,000 cfs.

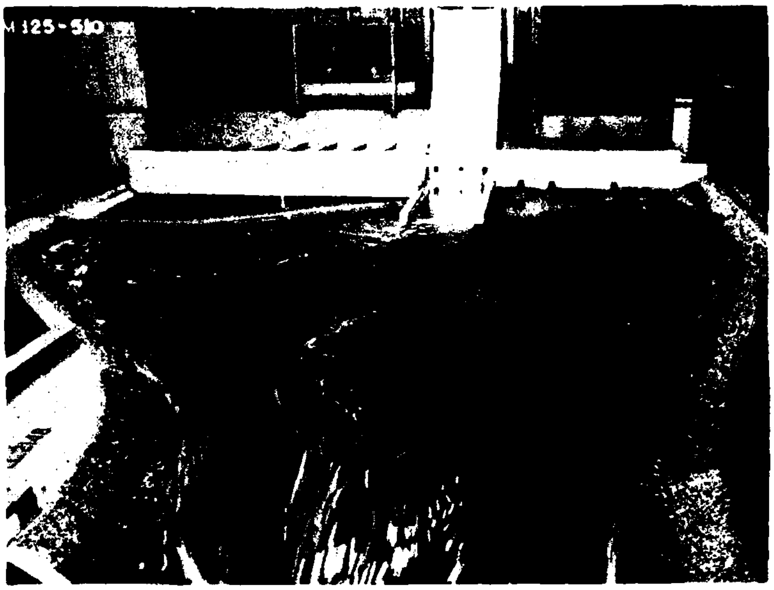


Photograph 20. Scour and deposition at stilling basin after 10 hours of flow with 82,000 cfs during second-step, second-stage diversion.

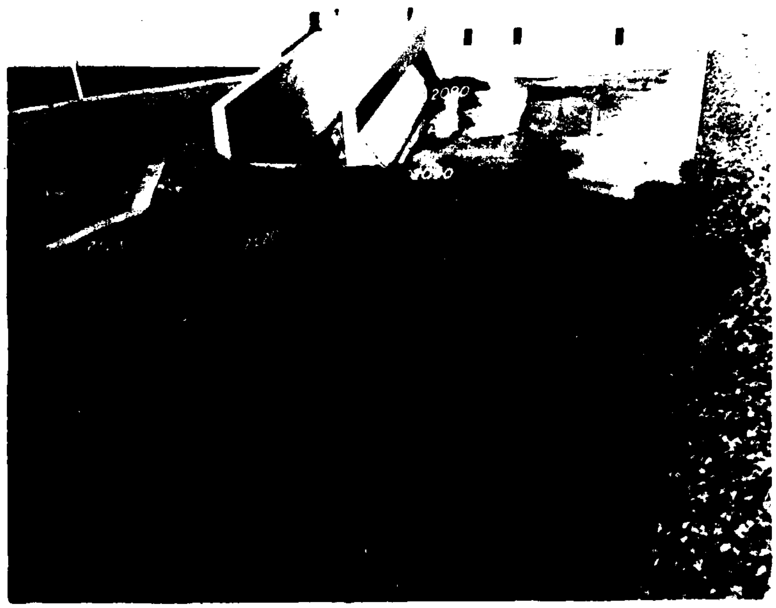
Photograph 21. Scour and deposition at stilling basin after 10 hours of flow with 109,000 cfs during second-step, second-stage diversion.



Photograph 22. Flow conditions during third-stage diversion. Discharge 20,000 cfs.



Flow conditions



Scour and deposition after 10 hours

Photograph 23. Third-stage diversion. Discharge 30,000

View from upstream

Discharge 109,000 cfs

Discharge 130,000 cfs

Debris caught on deflector piers and tower

Photograph 24. Low monoliths 33 to 35 and contractor's debris deflectors and tower, first-step, second-stage diversion.

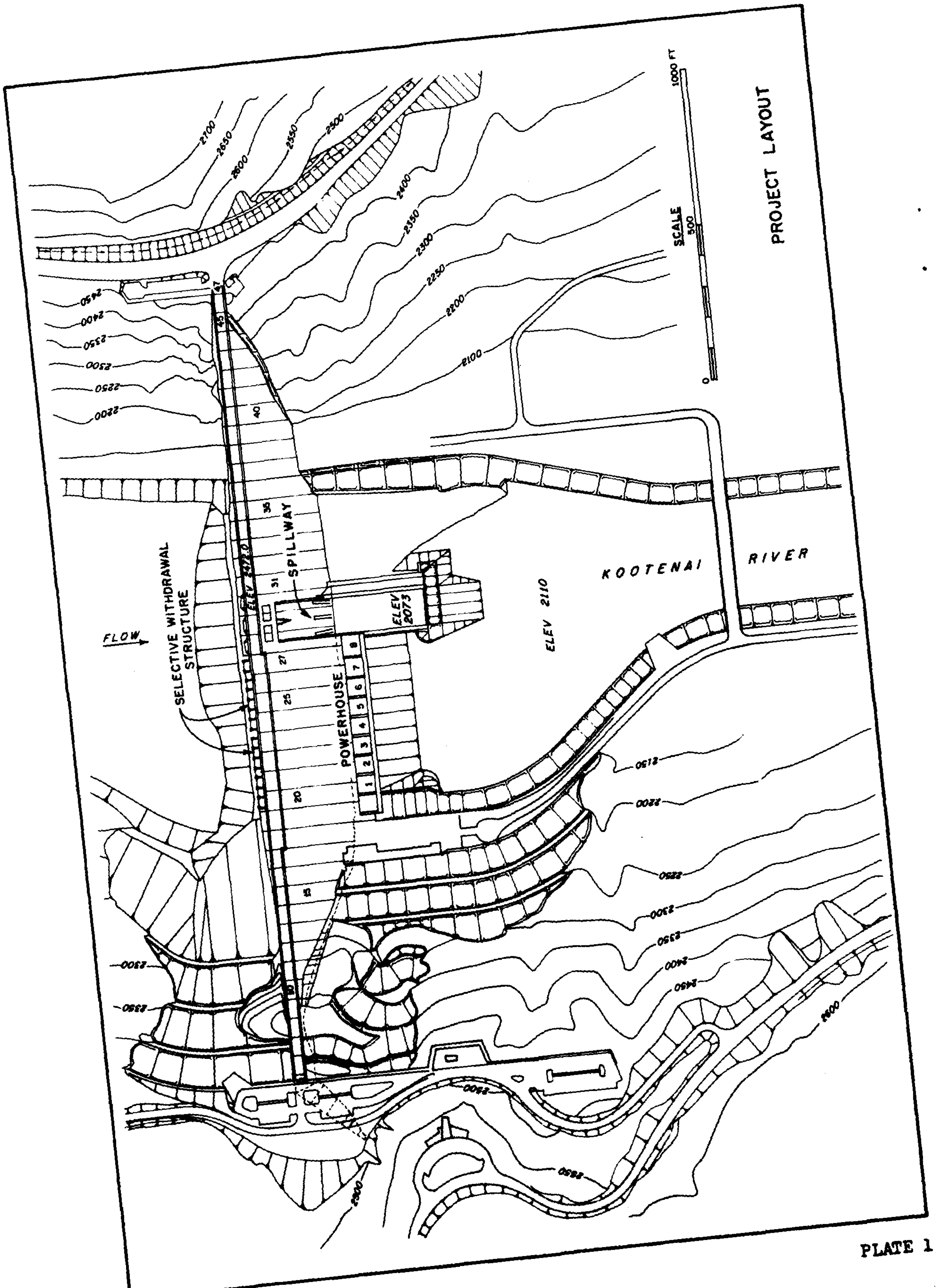
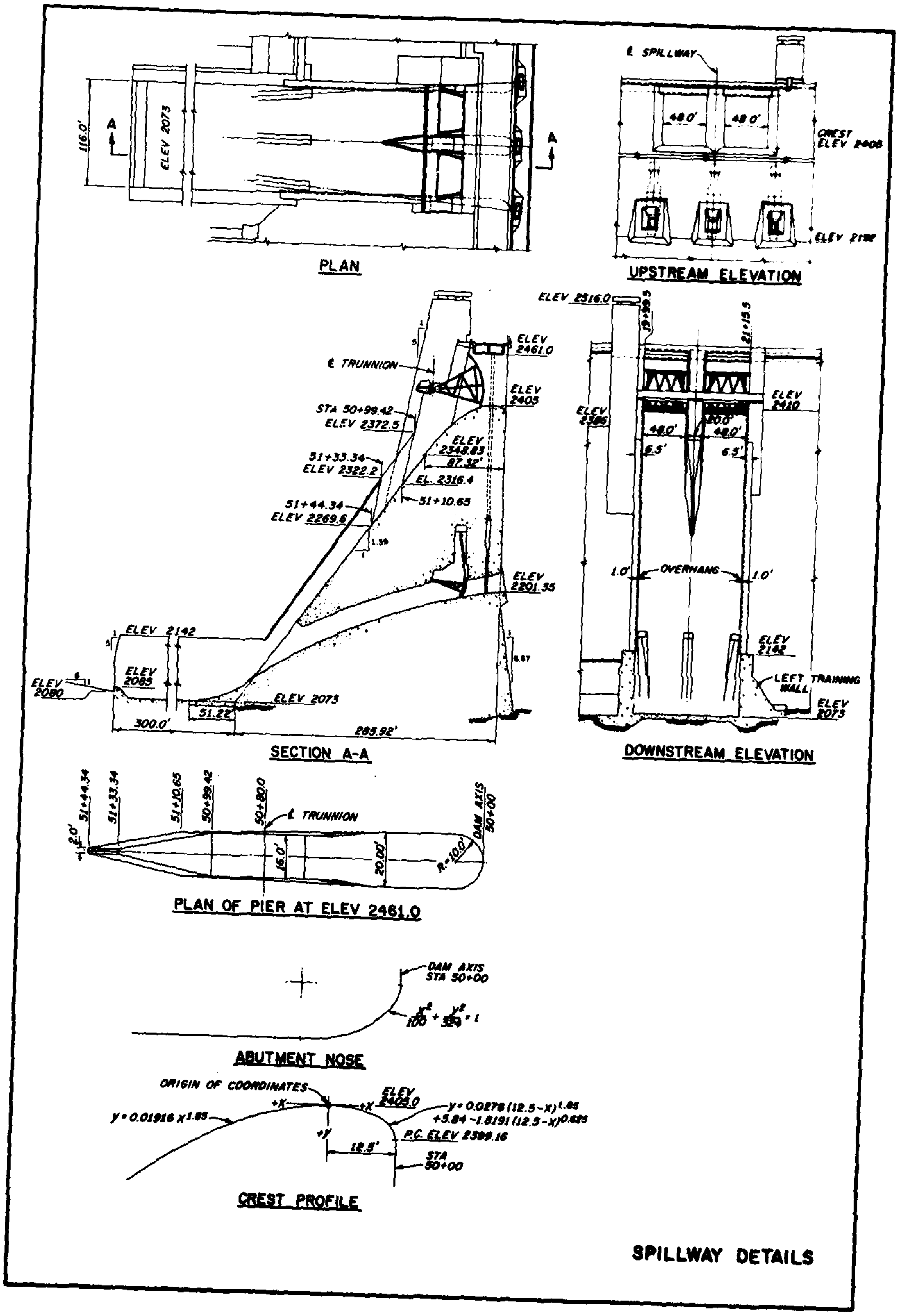
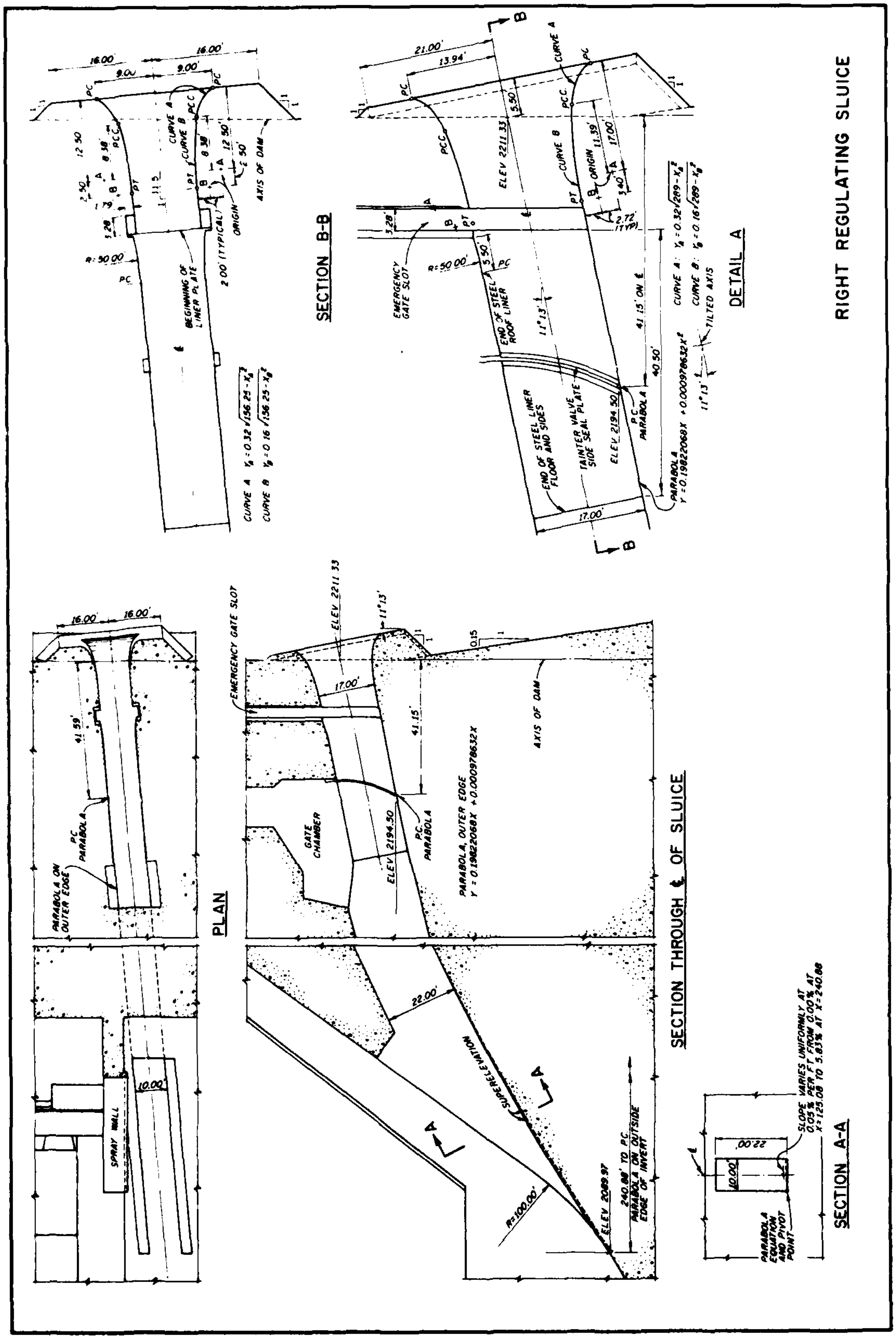

PROJECT LAYOUT
SCALE
0
500
1000 FT
KOOTENAI RIVER
ELEV 2110
ELEV 2073
SPILLWAY
POWERHOUSE
SELECTIVE WITHDRAWAL STRUCTURE
FLOW

PLATE 1



SPILLWAY DETAILS

PLATE 2


RIGHT REGULATING SLUICE
SECTION B-B
DETAIL A
PLAN
SECTION THROUGH ℄ OF SLUICE
SECTION A-A
EMERGENCY GATE SLOT
GATE CHAMBER
AXIS OF DAM
SPRAY WALL
SUPERELEVATION
ELEV 2211.33
ELEV 2194.50
ELEV 2089.97
PARABOLA, OUTER EDGE Y = 0.19822068X + 0.000978632X²
CURVE A $Y_A = 0.32\sqrt{156.25 - X_A^2}$
CURVE B $Y_B = 0.16\sqrt{156.25 - X_B^2}$
CURVE A: $Y_A = 0.32\sqrt{289 - X_A^2}$
CURVE B: $Y_B = 0.16\sqrt{289 - X_B^2}$
BEGINNING OF LINER PLATE
END OF STEEL ROOF LINER
END OF STEEL LINER FLOOR AND SIDES
TAINTER VALVE SIDE SEAL PLATE
TILTED AXIS
SLOPE VARIES UNIFORMLY AT 0.05% PER FT FROM 0.00% AT X=125.08 TO 5.83% AT X=240.88
PARABOLA EQUATION AND PIVOT POINT

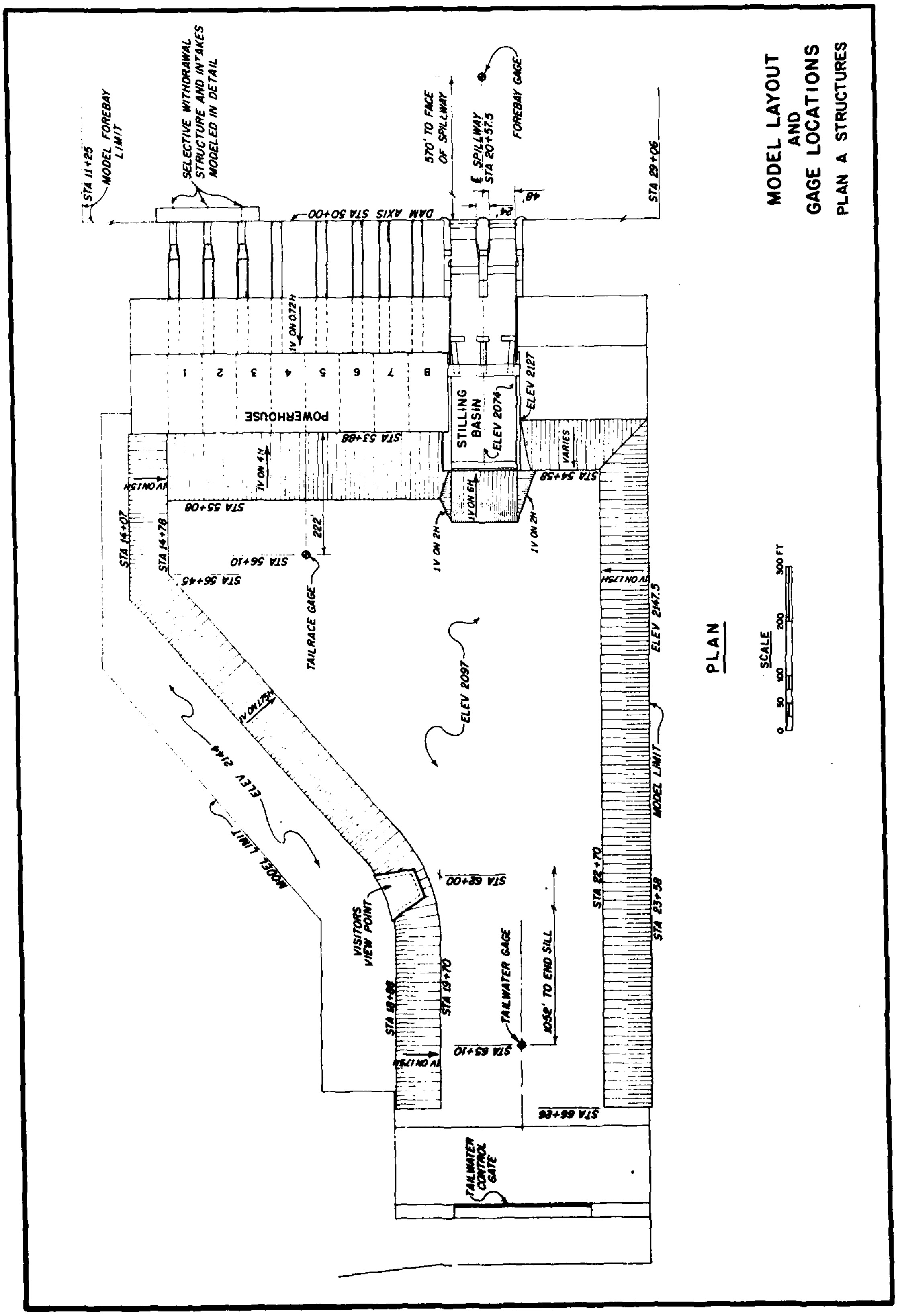

MODEL LAYOUT
AND
GAGE LOCATIONS
PLAN A STRUCTURES
PLAN
SCALE
0 50 100 200 300 FT
STA 11+25
MODEL FOREBAY LIMIT
SELECTIVE WITHDRAWAL STRUCTURE AND INTAKES MODELED IN DETAIL
570' TO FACE OF SPILLWAY
℄ SPILLWAY STA 20+57.5
FOREBAY GAGE
STA 29+06
DAM AXIS STA 50+00
24'
48'
1V ON 0.72H
POWERHOUSE
1 2 3 4 5 6 7 8
STILLING BASIN
ELEV 2074
ELEV 2127
STA 53+88
VARIES
STA 54+58
1V ON 4H
1V ON 6H
1V ON 2H
1V ON 15H
STA 55+08
222'
STA 56+10
STA 14+07
STA 14+78
STA 56+45
TAILRACE GAGE
1V ON 17.5H
ELEV 2147.5
ELEV 2097
1V ON 1.75H
ELEV 2144
MODEL LIMIT
VISITORS VIEW POINT
STA 62+00
STA 22+70
STA 23+58
STA 18+88
STA 19+70
TAILWATER GAGE
1052' TO END SILL
STA 65+10
STA 66+26
TAILWATER CONTROL GATE

PLATE 4

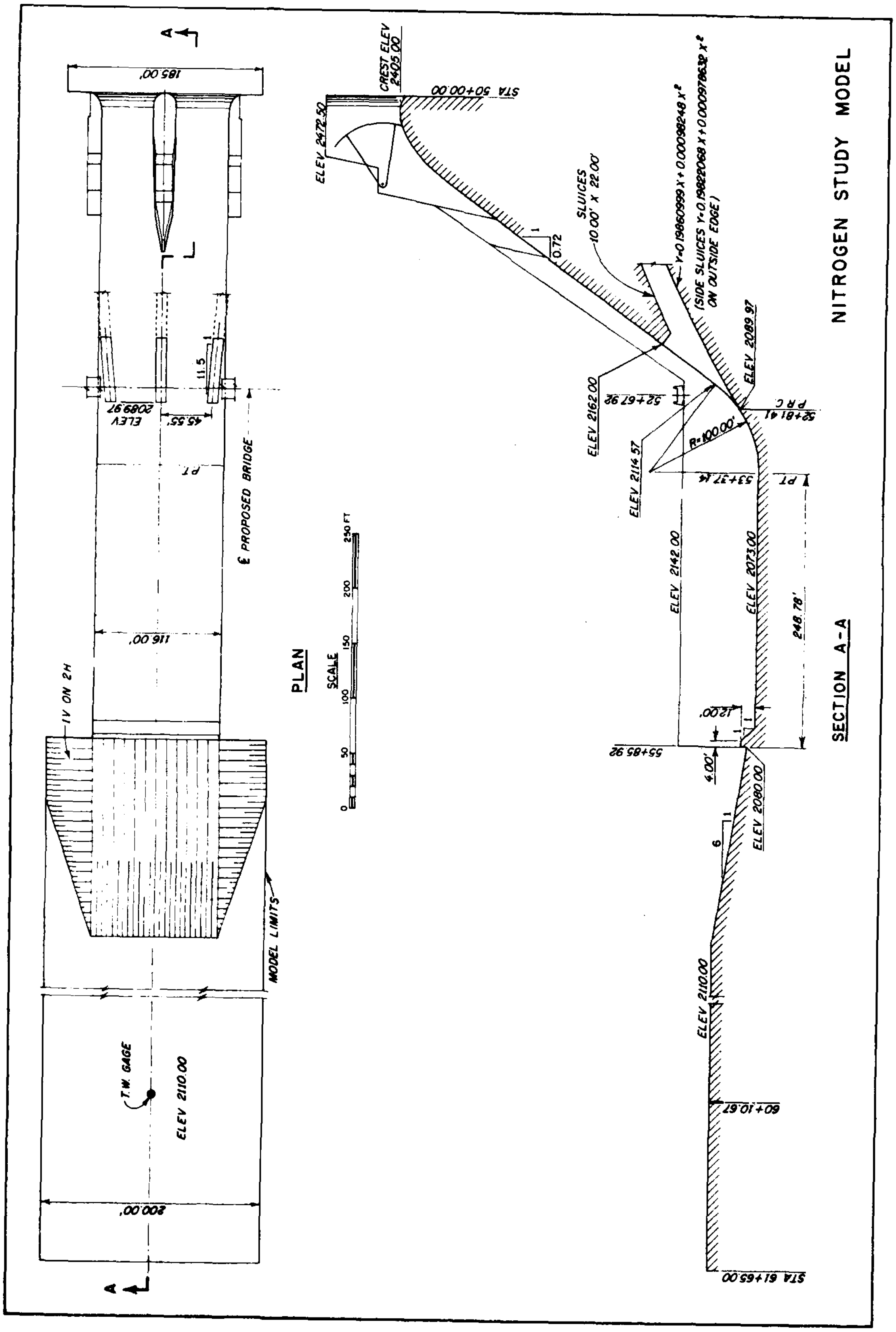
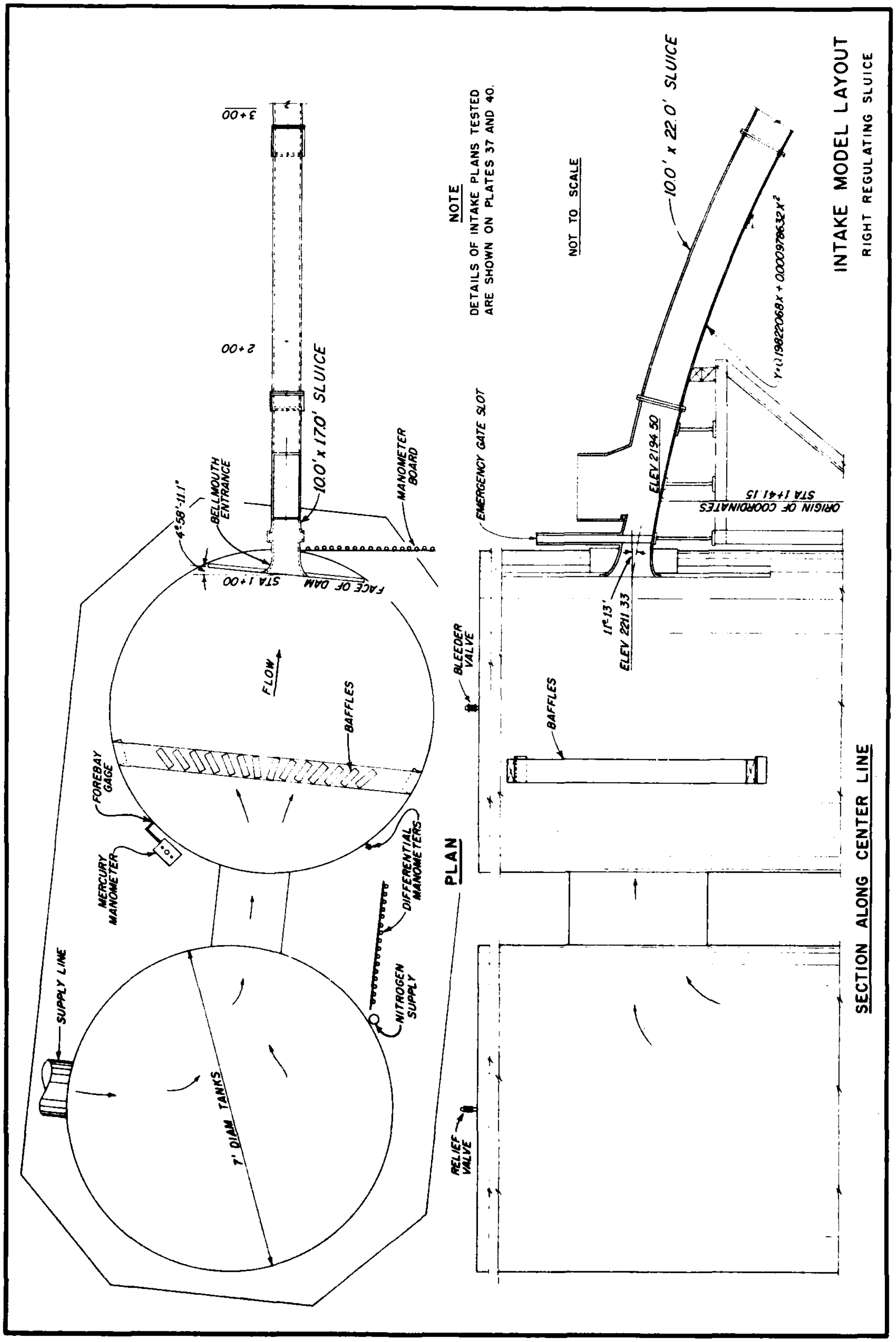
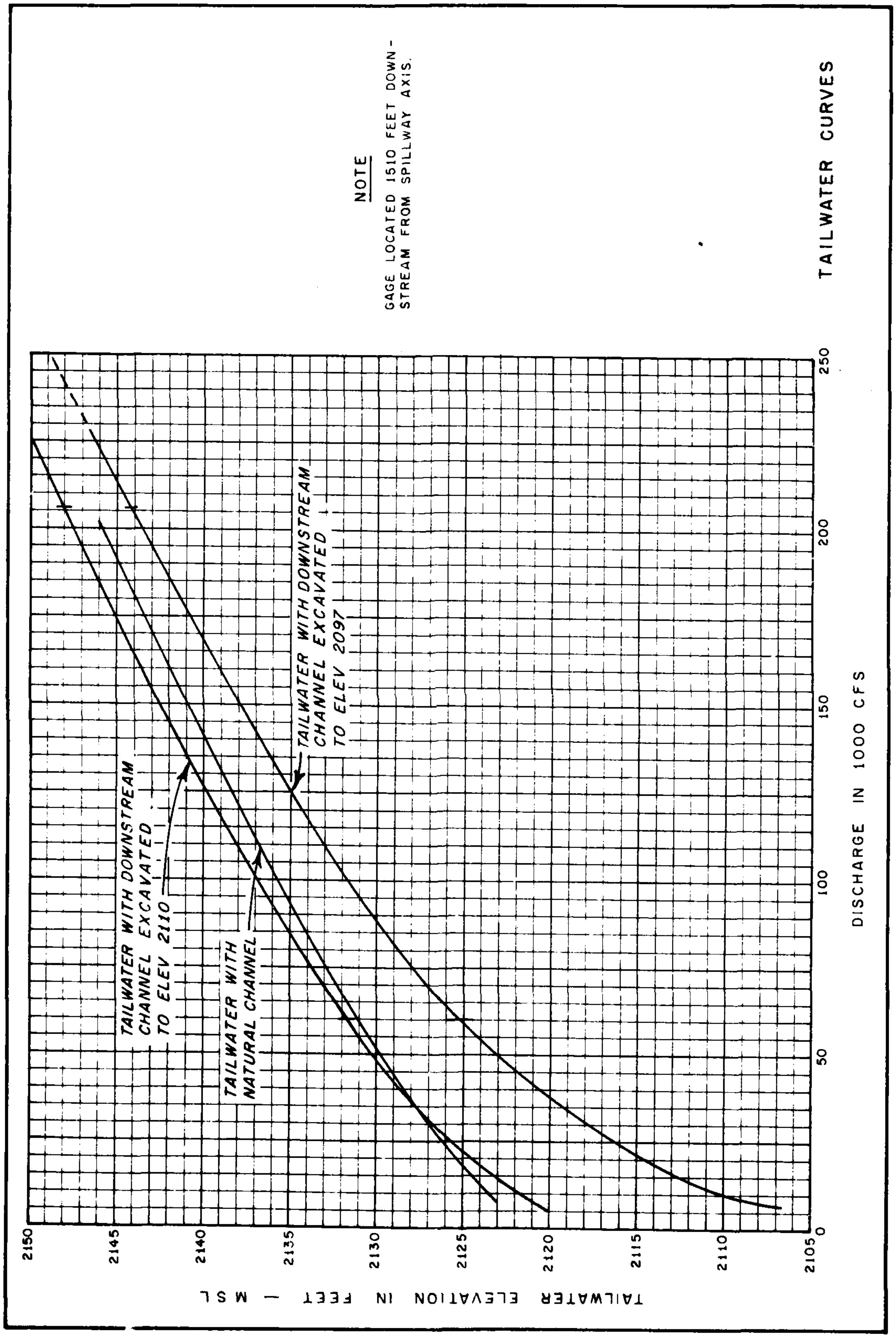
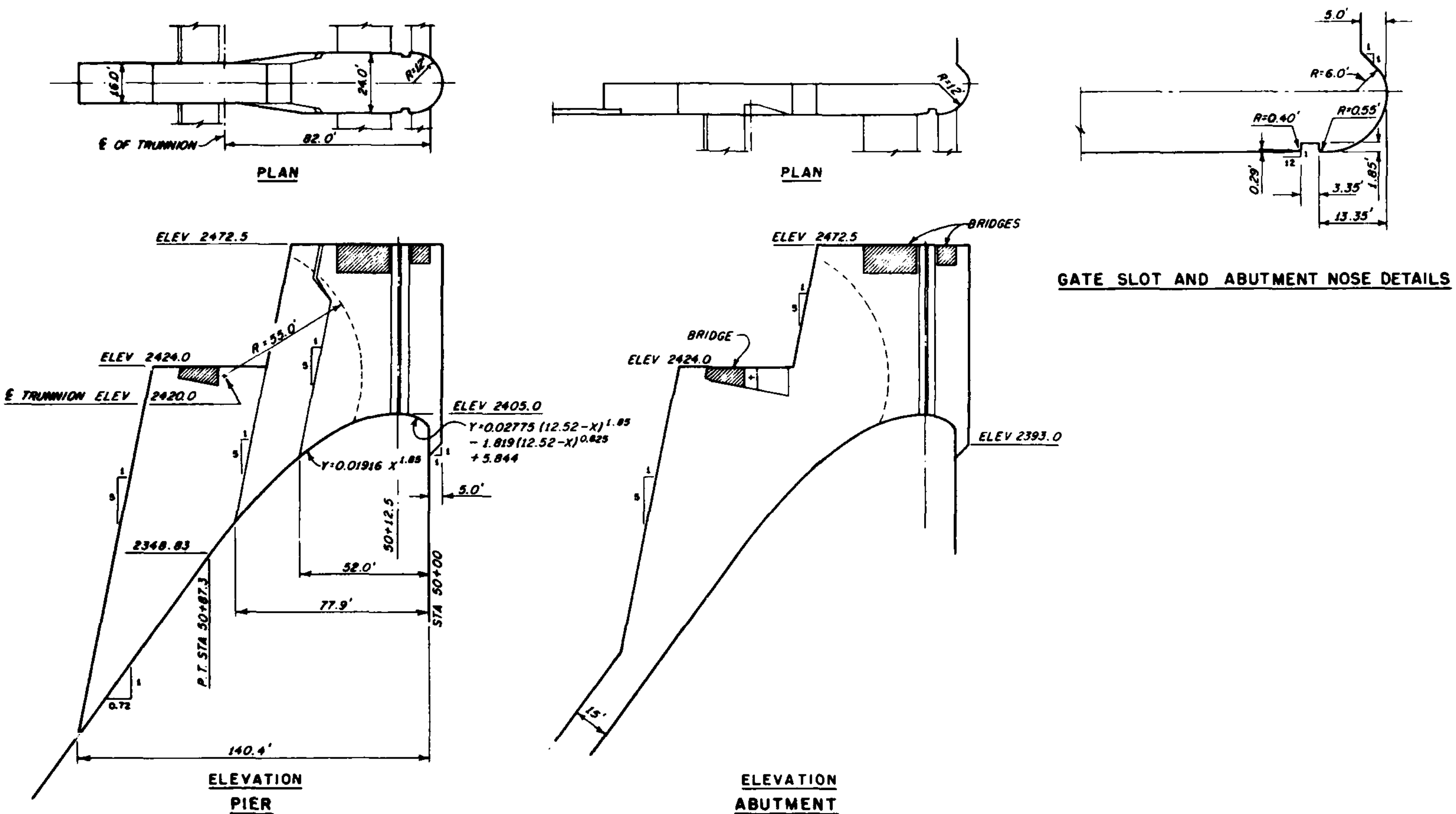

NITROGEN STUDY MODEL
PLAN
SCALE
0
50
100
150
200
250 FT
185.00'
ELEV 2089.97
45.55'
11.5
1
PT
℄ PROPOSED BRIDGE
116.00'
1V ON 2H
MODEL LIMITS
T.W. GAGE
ELEV 2110.00
200.00'
A
SECTION A-A
CREST ELEV 2405.00
STA 50+00.00
ELEV 2472.50
SLUICES 10.00' X 22.00'
1
0.72
Y=0.19860999 X + 0.00098248 X²
(SIDE SLUICES Y=0.19822068 X + 0.00097863 X² ON OUTSIDE EDGE)
ELEV 2089.97
ELEV 2162.00
52+67.92
52+81.41
P.R.C.
R=100.00'
ELEV 2114.57
53+37.14
PT
ELEV 2142.00
ELEV 2073.00
248.78'
12.00'
4.00'
55+85.92
ELEV 2080.00
6
ELEV 2110.00
60+10.67
STA 61+65.00

PLATE 5


INTAKE MODEL LAYOUT
RIGHT REGULATING SLUICE
NOTE
DETAILS OF INTAKE PLANS TESTED ARE SHOWN ON PLATES 37 AND 40.
NOT TO SCALE
10.0' x 22.0' SLUICE
3+00
2+00
10.0' x 17.0' SLUICE
BELLMOUTH ENTRANCE
MANOMETER BOARD
4°58'-11.1"
STA 1+00
FACE OF DAM
FLOW
BAFFLES
FOREBAY GAGE
MERCURY MANOMETER
DIFFERENTIAL MANOMETERS
NITROGEN SUPPLY
SUPPLY LINE
7' DIAM TANKS
PLAN
EMERGENCY GATE SLOT
ELEV 2194.50
STA 1+41.15
ORIGIN OF COORDINATES
11°-13'
ELEV 2211.33
BLEEDER VALVE
BAFFLES
RELIEF VALVE
SECTION ALONG CENTER LINE

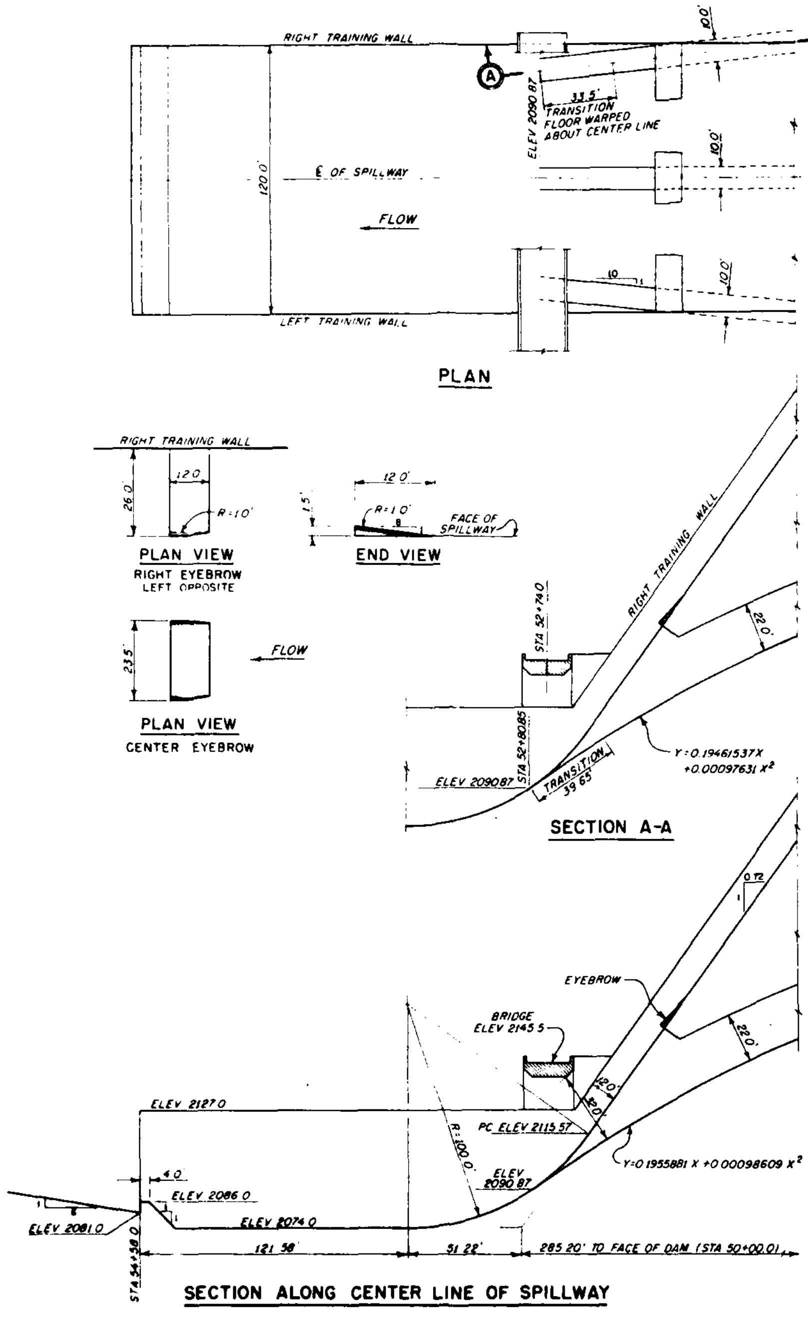


TAILWATER CURVES
NOTE
GAGE LOCATED 1510 FEET DOWN-STREAM FROM SPILLWAY AXIS.
TAILWATER WITH DOWNSTREAM CHANNEL EXCAVATED TO ELEV 2097
TAILWATER WITH DOWNSTREAM CHANNEL EXCAVATED TO ELEV 2110
TAILWATER WITH NATURAL CHANNEL
DISCHARGE IN 1000 CFS
0
50
100
150
200
250
TAILWATER ELEVATION IN FEET — MSL
2105
2110
2115
2120
2125
2130
2135
2140
2145
2150



DETAILS

PLAN A CREST, PIER, AND ABUTMENTS

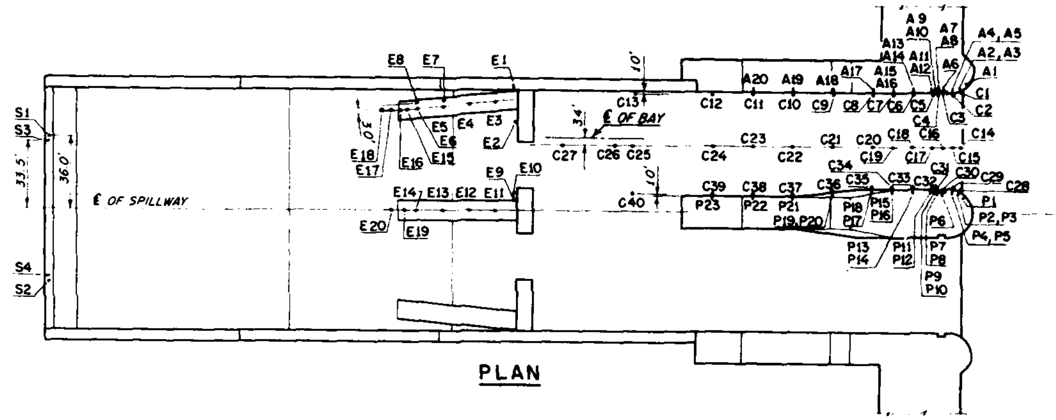


NOTE

DETAILS OF CREST, PIER, AND ABUTMENTS ARE SHOWN ON PLATE 8

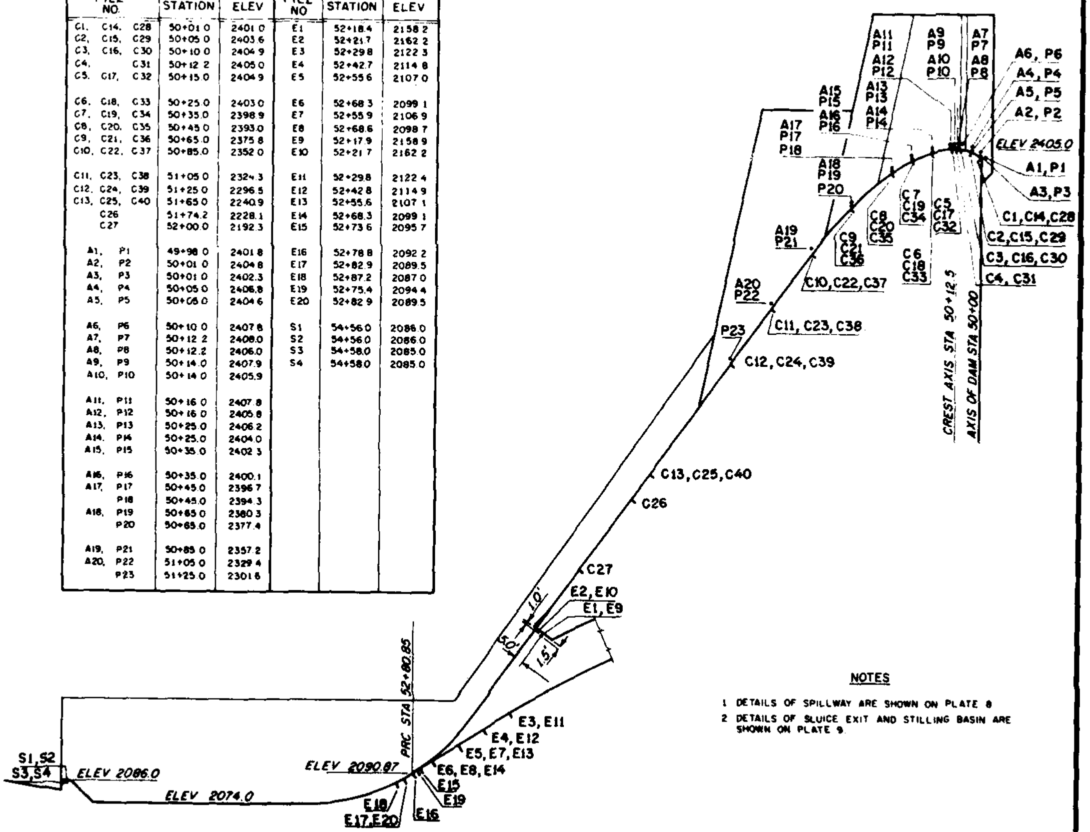
DETAILS

SLUICE EXITS AND STILLING BASIN
PLAN A STRUCTURES



PLAN

PIEZOMETER LOCATIONS					
PIEZ NO.	STATION	ELEV	PIEZ NO	STATION	ELEV
C1, C14, C28	50+01.0	2401.0	E1	52+18.4	2158.2
C2, C15, C29	50+05.0	2403.6	E2	52+21.7	2162.2
C3, C16, C30	50+10.0	2404.9	E3	52+29.8	2122.3
C4, C31	50+12.2	2405.0	E4	52+42.7	2114.8
C5, C17, C32	50+15.0	2404.9	E5	52+55.6	2107.0
C6, C18, C33	50+25.0	2403.0	E6	52+68.3	2099.1
C7, C19, C34	50+35.0	2398.9	E7	52+55.9	2106.9
C8, C20, C35	50+45.0	2393.0	E8	52+68.6	2098.7
C9, C21, C36	50+65.0	2375.8	E9	52+17.9	2158.9
C10, C22, C37	50+85.0	2352.0	E10	52+21.7	2162.2
C11, C23, C38	51+05.0	2324.3	E11	52+29.8	2122.4
C12, C24, C39	51+25.0	2296.5	E12	52+42.8	2114.9
C13, C25, C40	51+65.0	2240.9	E13	52+55.6	2107.1
C26	51+74.2	2228.1	E14	52+68.3	2099.1
C27	52+00.0	2192.3	E15	52+73.6	2095.7
A1, P1	49+98.0	2401.8	E16	52+78.8	2092.2
A2, P2	50+01.0	2404.8	E17	52+82.9	2089.5
A3, P3	50+01.0	2402.3	E18	52+87.2	2087.0
A4, P4	50+05.0	2406.8	E19	52+75.4	2094.4
A5, P5	50+05.0	2404.6	E20	52+82.9	2089.5
A6, P6	50+10.0	2407.8	S1	54+56.0	2086.0
A7, P7	50+12.2	2408.0	S2	54+56.0	2086.0
A8, P8	50+12.2	2406.0	S3	54+58.0	2085.0
A9, P9	50+14.0	2407.9	S4	54+58.0	2085.0
A10, P10	50+14.0	2405.9			
A11, P11	50+16.0	2407.8			
A12, P12	50+16.0	2405.8			
A13, P13	50+25.0	2406.2			
A14, P14	50+25.0	2404.0			
A15, P15	50+35.0	2402.3			
A16, P16	50+35.0	2400.1			
A17, P17	50+45.0	2396.7			
P18	50+45.0	2394.3			
A18, P19	50+65.0	2380.3			
P20	50+65.0	2377.4			
A19, P21	50+85.0	2357.2			
A20, P22	51+05.0	2329.4			
P23	51+25.0	2301.6			



SECTION ALONG CENTER LINE

SCALE

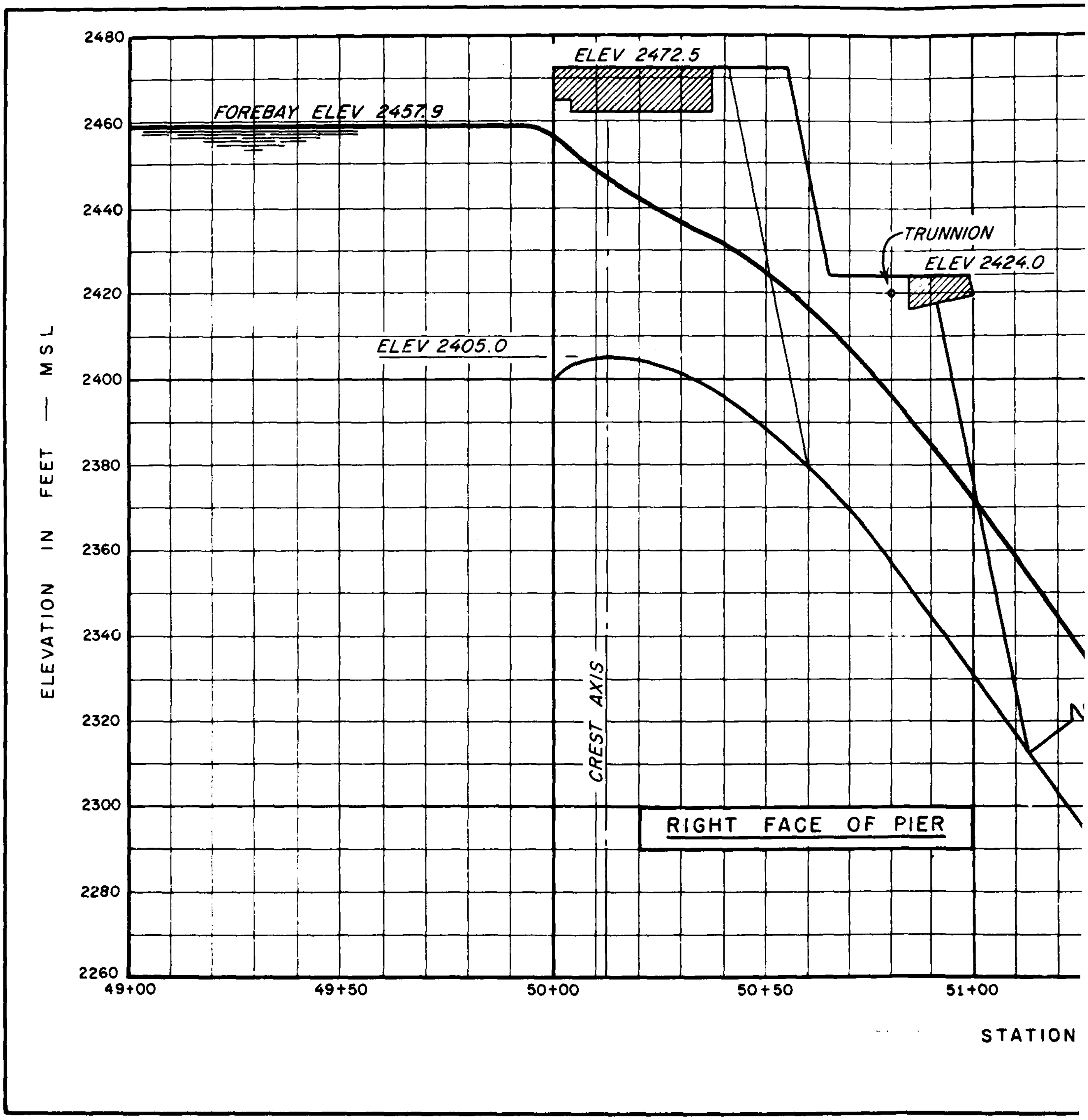
0 25 50 75 100 FT

NOTES

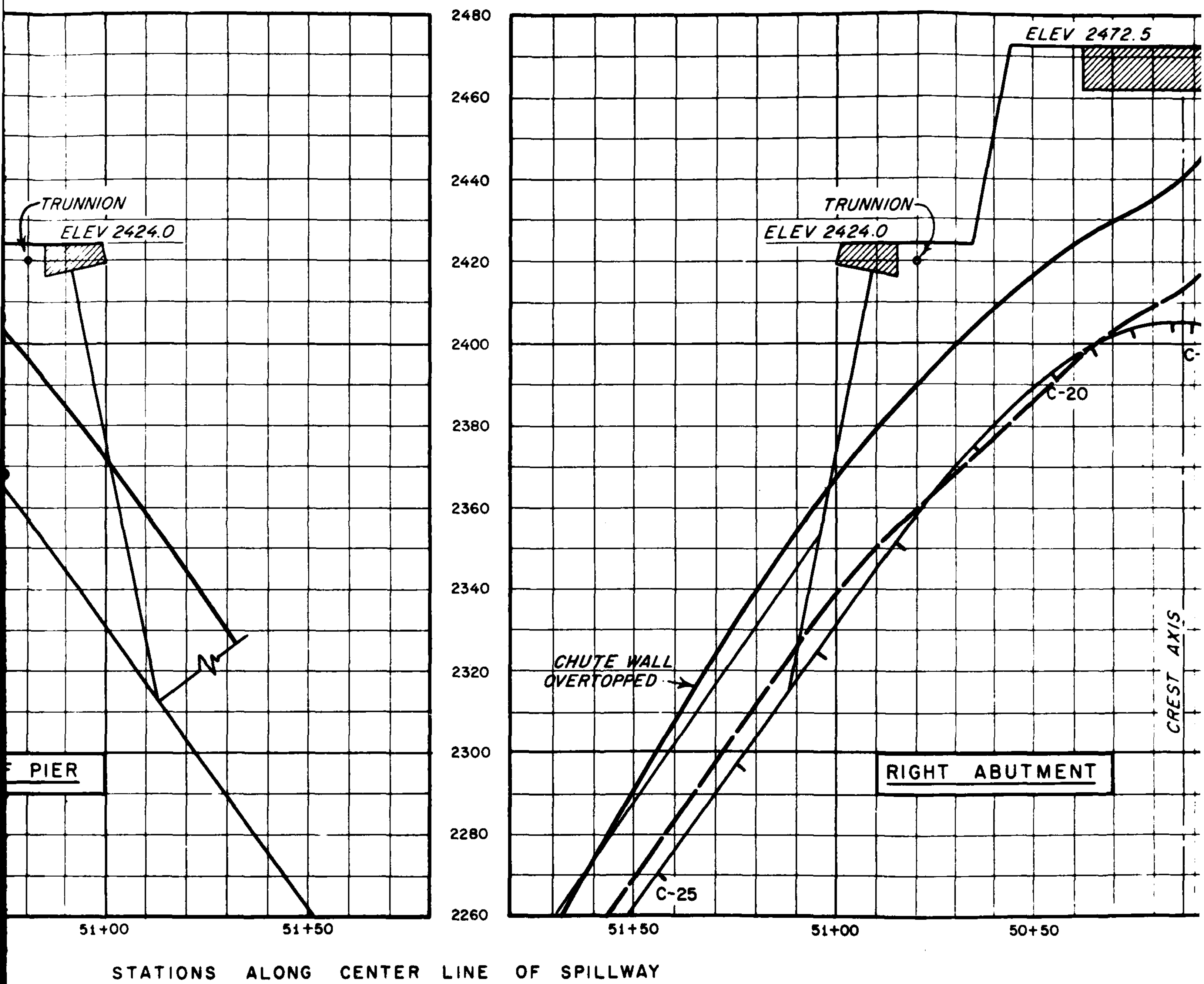
1 DETAILS OF SPILLWAY ARE SHOWN ON PLATE 8
2 DETAILS OF SLUICE EXIT AND STILLING BASIN ARE SHOWN ON PLATE 9.

PIEZOMETER LOCATIONS

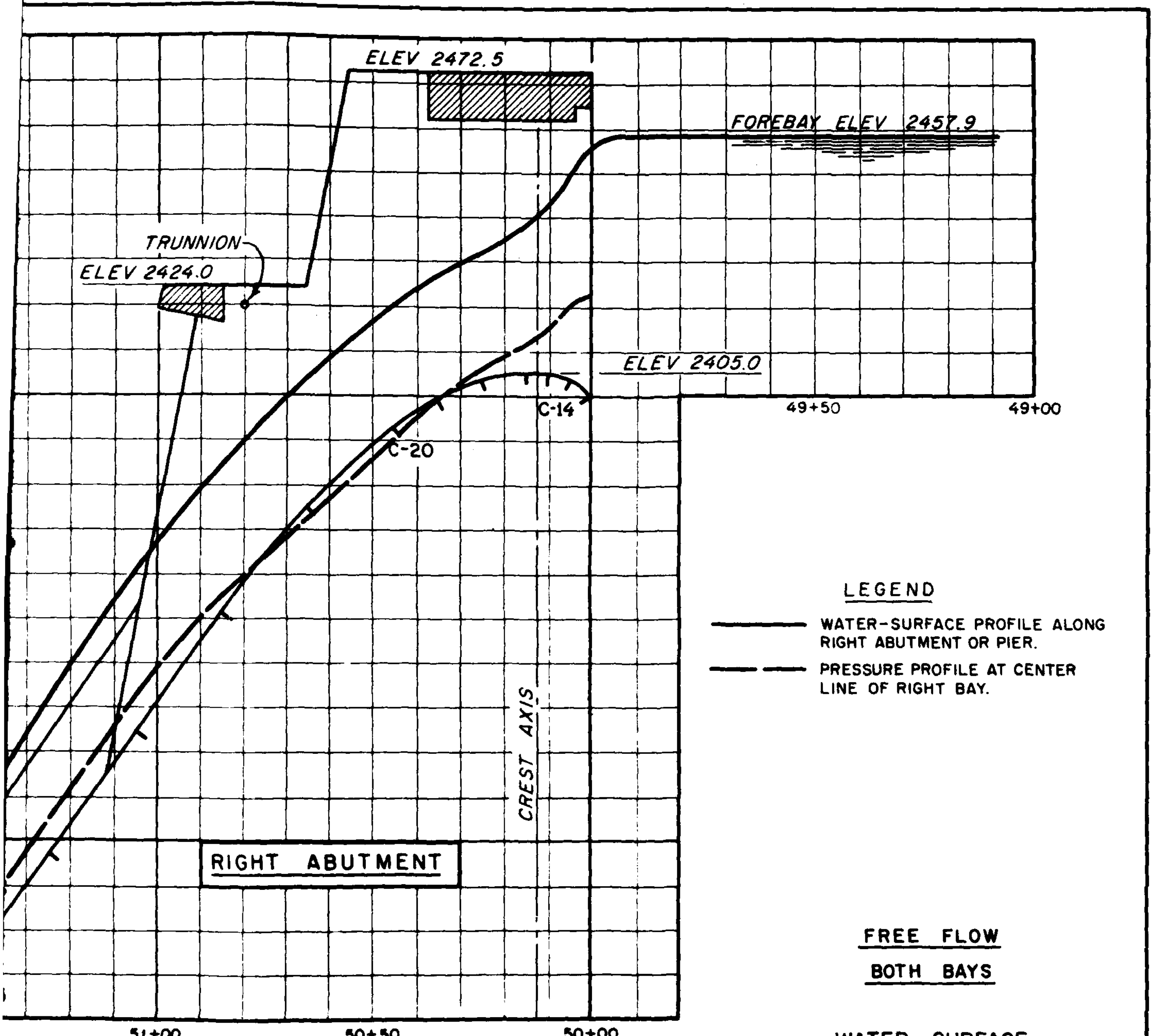
PLAN A SPILLWAY



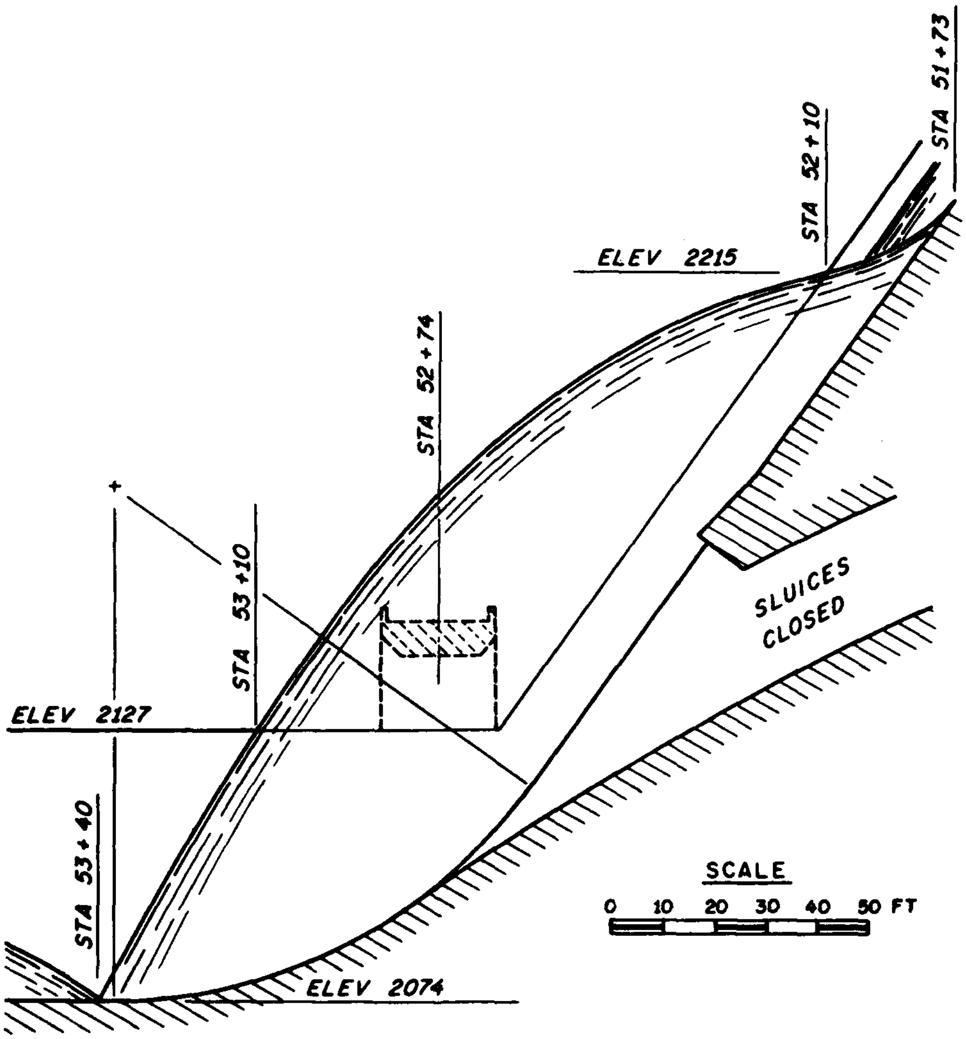
ELEV 2472.5
FOREBAY ELEV 2457.9
TRUNNION
ELEV 2424.0
ELEV 2405.0
CREST AXIS
RIGHT FACE OF PIER
ELEVATION IN FEET — MSL
2480
2460
2440
2420
2400
2380
2360
2340
2320
2300
2280
2260
49+00
49+50
50+00
50+50
51+00
STATION


TRUNNION
ELEV 2424.0
PIER
51+00
51+50
2480
2460
2440
2420
2400
2380
2360
2340
2320
2300
2280
2260
ELEV 2472.5
TRUNNION
ELEV 2424.0
C-20
CHUTE WALL
OVERTOPPED
RIGHT ABUTMENT
CREST AXIS
C-25
51+50
51+00
50+50
STATIONS ALONG CENTER LINE OF SPILLWAY

2



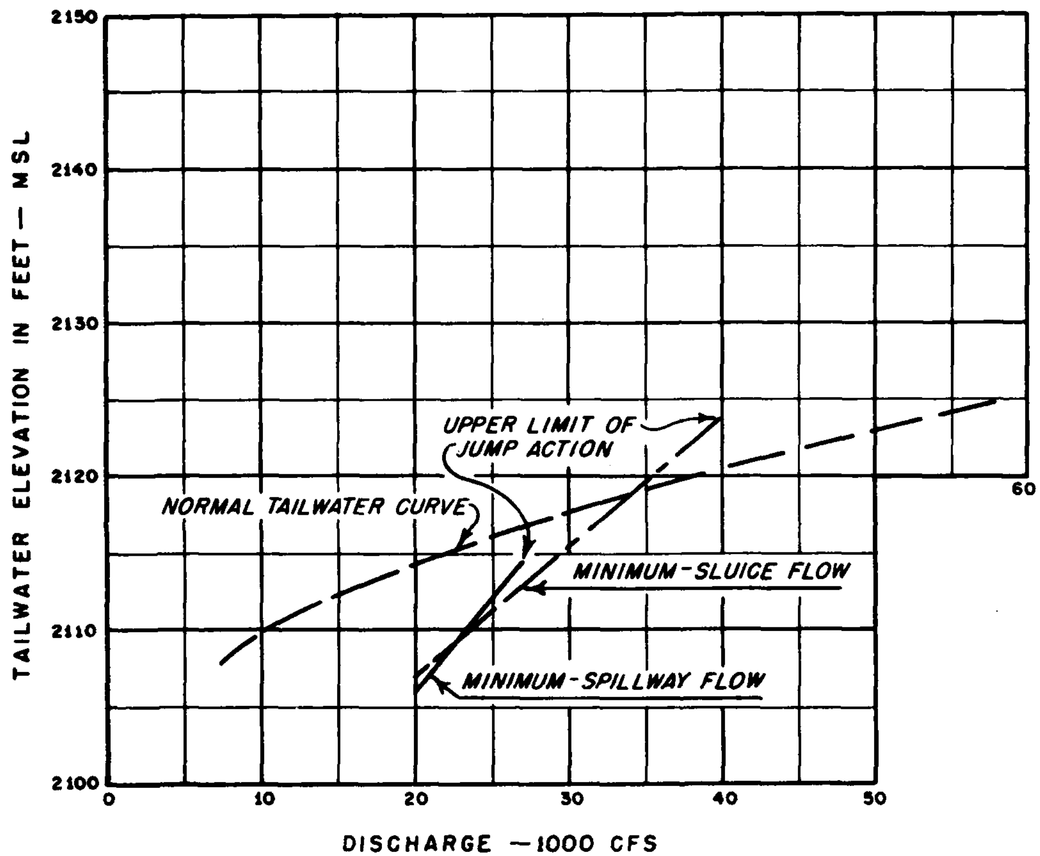
WATER - SURFACE
AND PRESSURE PROFILES
PLAN B SPILLWAY STRUCTURES
SPILLWAY DISCHARGE 145 000 CFS



SECTION ALONG CENTER LINE OF SPILLWAY

ROOSTER TAIL IN CHUTE

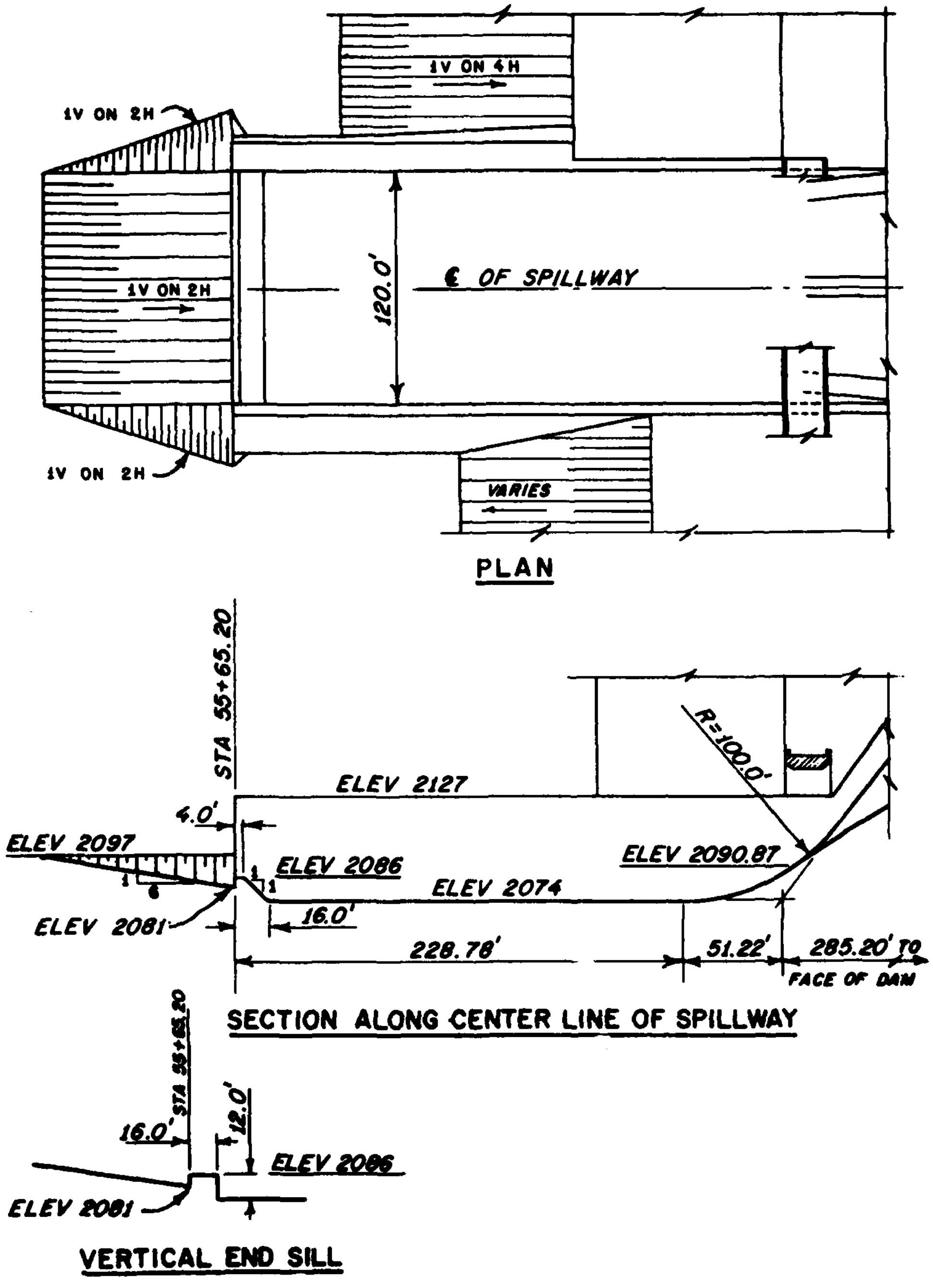
SPILLWAY DISCHARGE 145 000 CFS
PLAN A STRUCTURES



NOTES

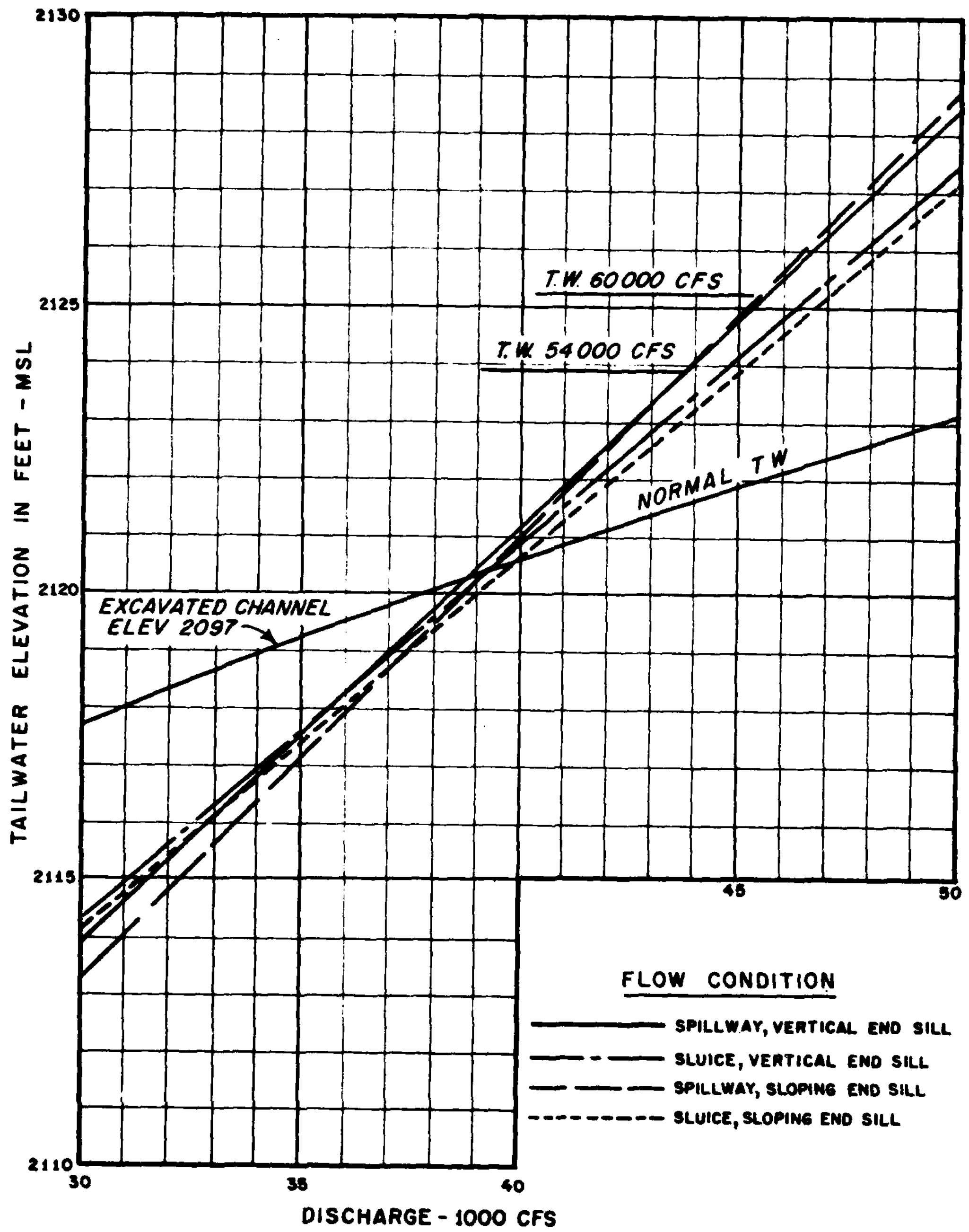
1. DETAILS OF STILLING BASIN SHOWN ON PLATE 9.
2. DETAILS OF CHANNEL AND LOCATION OF TAILWATER GAGE SHOWN ON PLATE 4.
3. FOREBAY ELEV 2459.

MINIMUM TAILWATER FOR HYDRAULIC JUMP
PLAN A STILLING BASIN
CHANNEL ELEV 2097



DETAILS

PLAN B STILLING BASIN

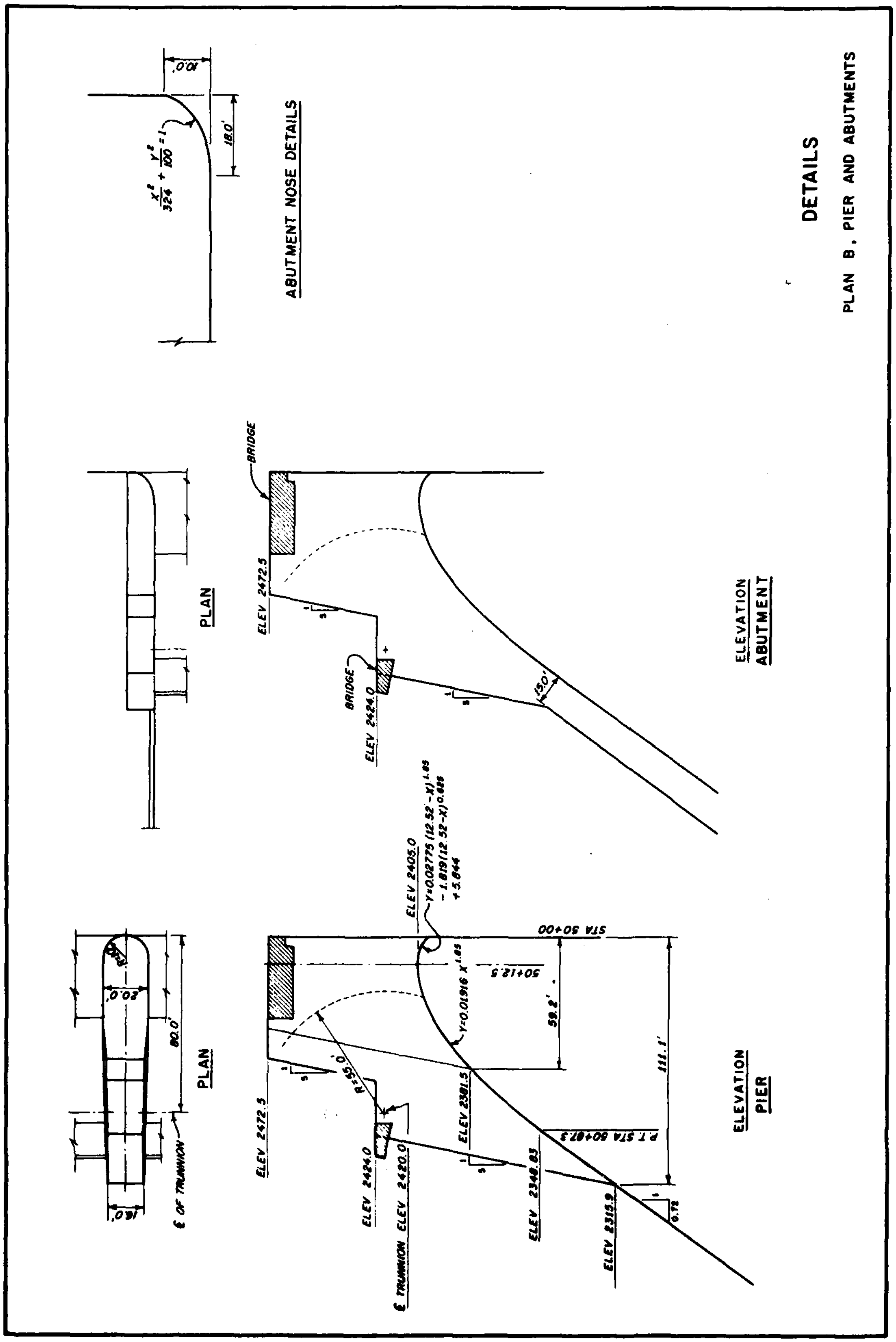


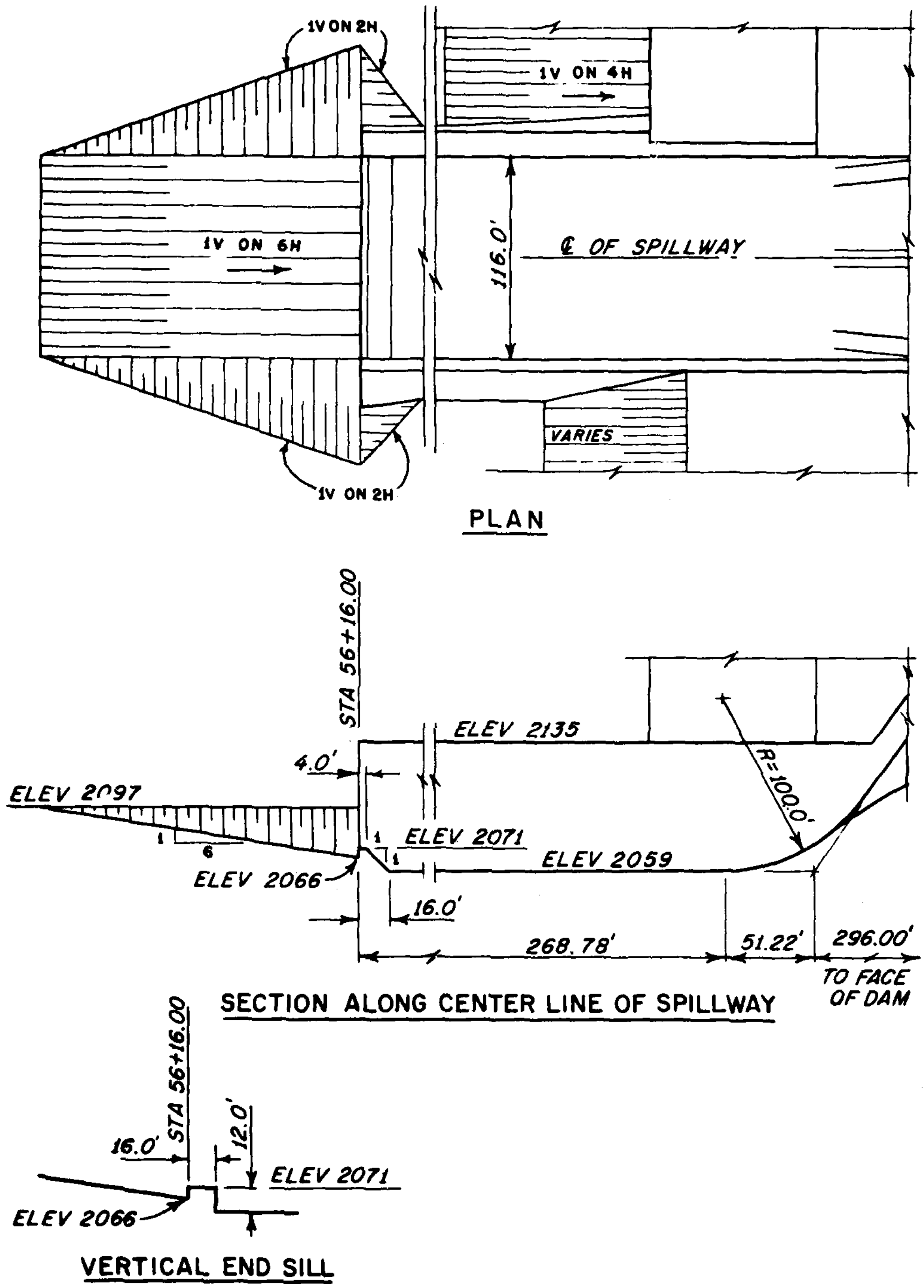
NOTES

1. DETAILS OF STILLING BASIN ARE SHOWN ON PLATE 14.
2. TAILWATER GAGE LOCATION IS SHOWN ON PLATE 4.
3. FOREBAY ELEVATION 2459.

MINIMUM TAILWATER FOR HYDRAULIC JUMP

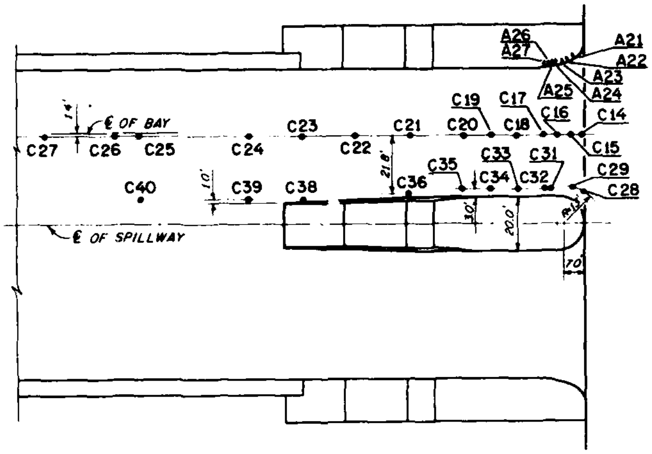
PLAN B STILLING BASIN


DETAILS
PLAN B, PIER AND ABUTMENTS
ABUTMENT NOSE DETAILS
$\frac{X^2}{324} + \frac{Y^2}{100} = 1$
10.0'
18.0'
PLAN
ELEVATION
ABUTMENT
BRIDGE
ELEV 2472.5
ELEV 2424.0
15.0'
ELEVATION
PIER
ELEV 2405.0
$Y = 0.02775(12.52-X)^{1.85} - 1.819(12.52-X)^{0.625} + 5.844$
STA 50+00
50+12.5
$Y = 0.01916X^{1.85}$
59.2'
111.1'
R=55.0
ELEV 2381.5
P.T. STA 50+87.3
ELEV 2348.83
ELEV 2319.9
0.78
℄ TRUNNION ELEV 2420.0
20.0'
80.0'
16.0'
℄ OF TRUNNION



DETAILS

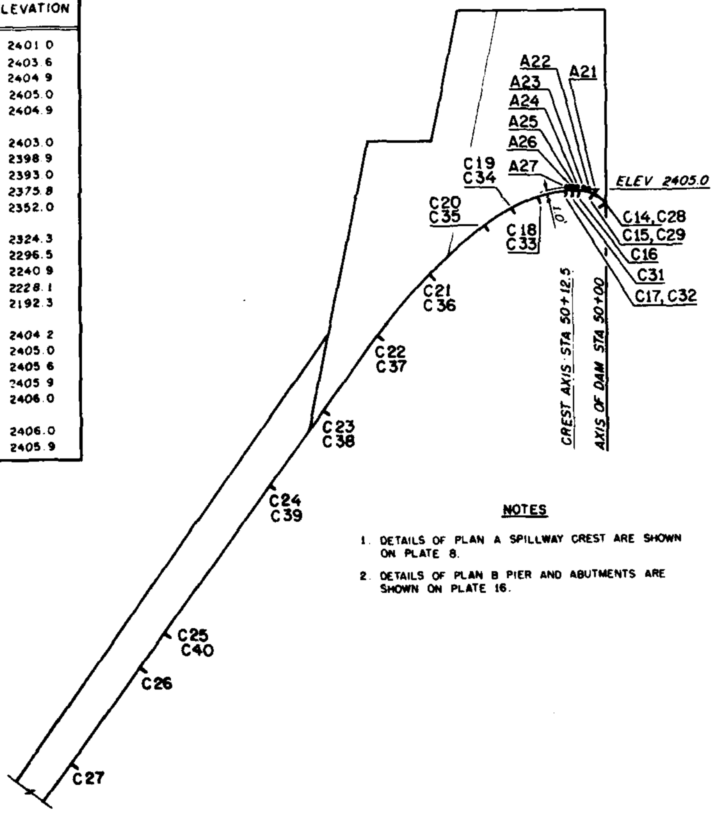
PLAN C STILLING BASIN



PLAN

PIEZOMETER LOCATIONS

PIEZOMETER NUMBER		STATION	ELEVATION
C14	C28	50+01.0	2401.0
C15	C29	50+05.0	2403.6
C16		50+10.0	2404.9
	C31	50+12.2	2405.0
C17	C32	50+15.0	2404.9
C18	C33	50+25.0	2403.0
C19	C34	50+35.0	2398.9
C20	C35	50+45.0	2393.0
C21	C36	50+65.0	2375.8
C22	C37	50+85.0	2352.0
C23	C38	51+05.0	2324.3
C24	C39	51+25.0	2296.5
C25	C40	51+65.0	2240.9
C26		51+74.2	2228.1
C27		52+00.0	2192.3
A21		50+03.8	2404.2
A22		50+06.2	2405.0
A23		50+08.2	2405.6
A24		50+10.6	2405.9
A25		50+12.0	2406.0
A26		50+13.5	2406.0
A27		50+15.0	2405.9



SECTION ALONG CENTER LINE

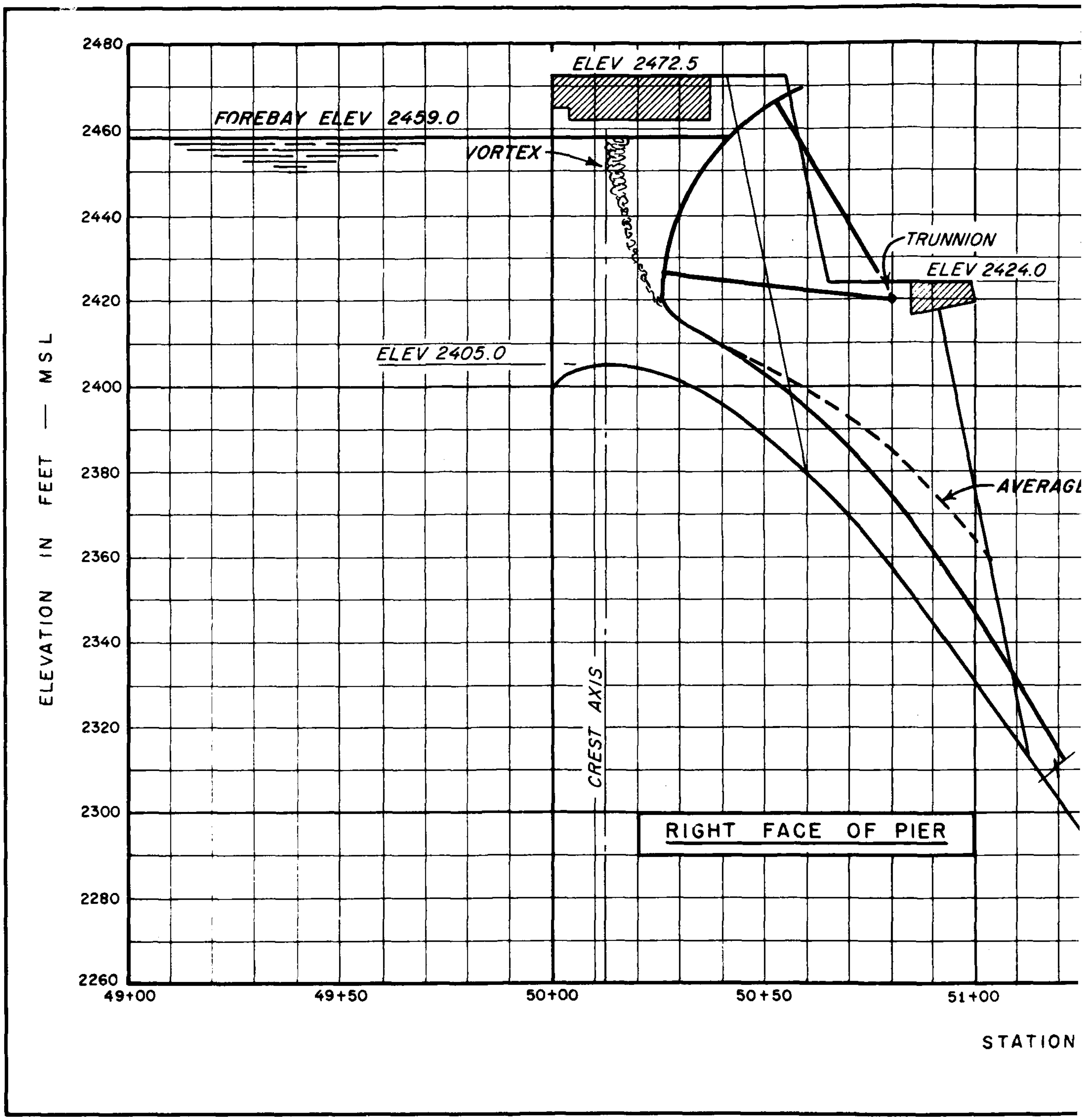
SCALE

0 25 50 75 100 FT

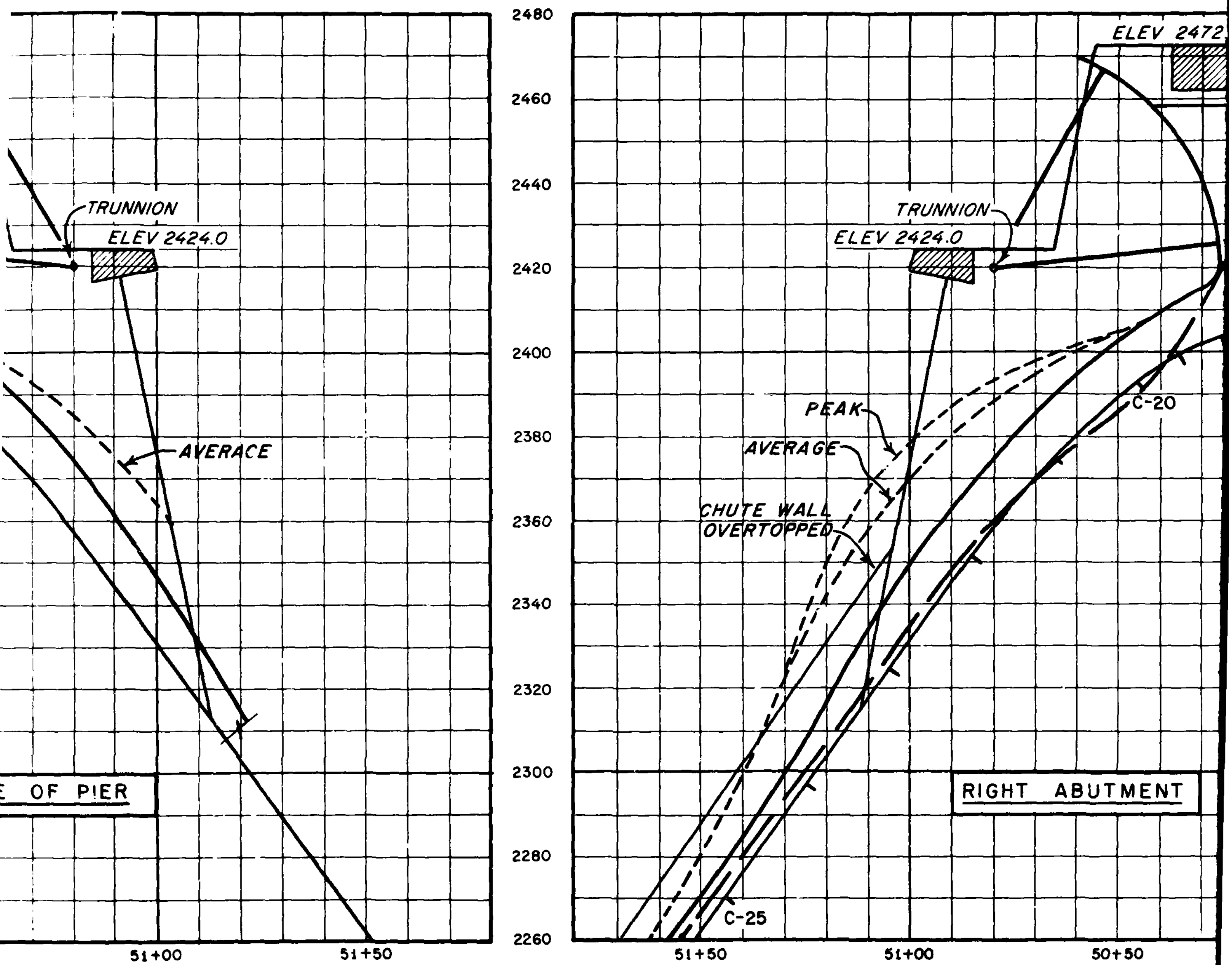
NOTES

1. DETAILS OF PLAN A SPILLWAY CREST ARE SHOWN ON PLATE 8.
2. DETAILS OF PLAN B PIER AND ABUTMENTS ARE SHOWN ON PLATE 16.

PIEZOMETER LOCATIONS
PLAN A CREST
PLAN B PIER AND ABUTMENTS

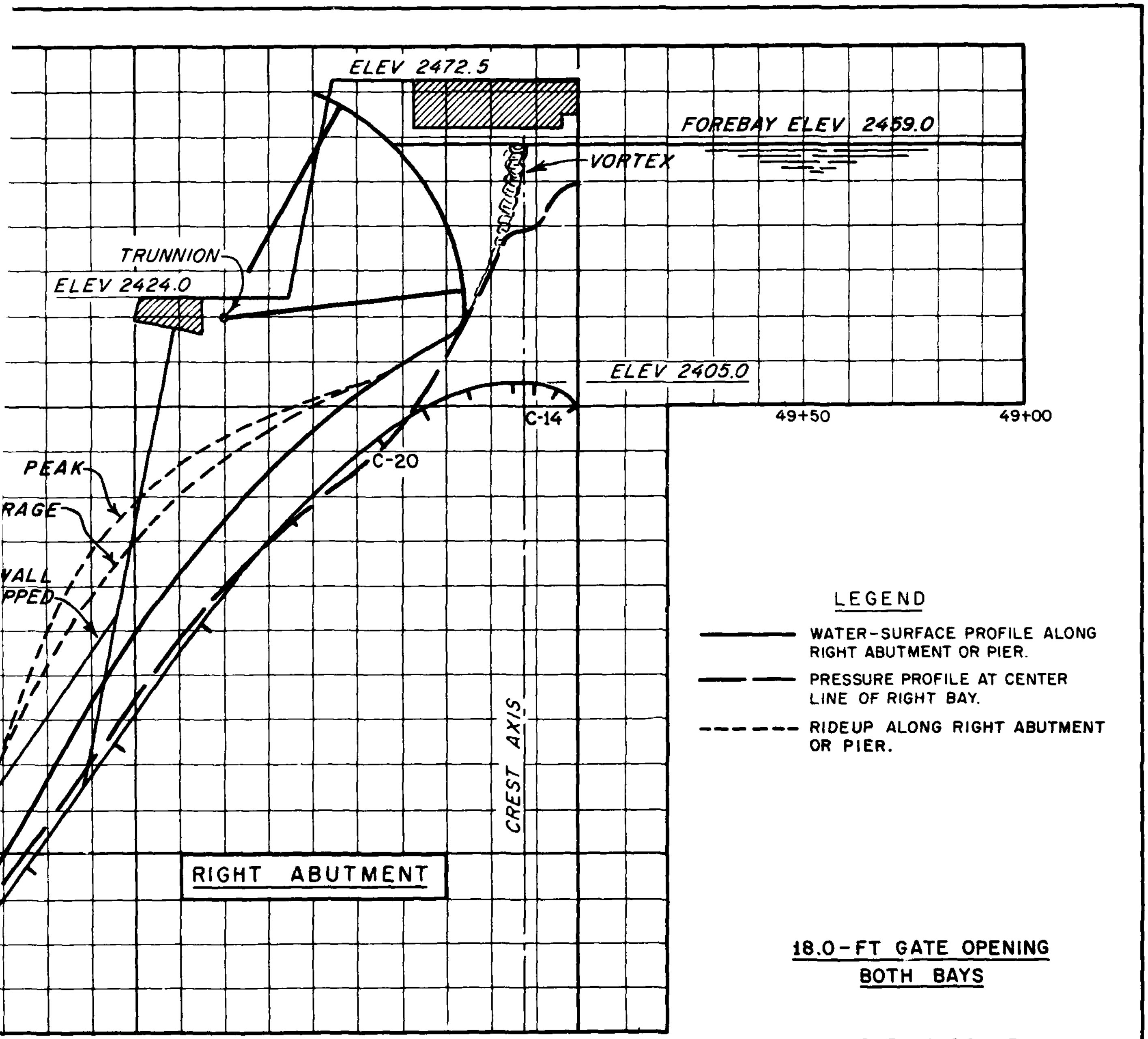


ELEV 2472.5
FOREBAY ELEV 2459.0
VORTEX
TRUNNION
ELEV 2424.0
ELEV 2405.0
AVERAGE
CREST AXIS
RIGHT FACE OF PIER
ELEVATION IN FEET — MSL
2480
2460
2440
2420
2400
2380
2360
2340
2320
2300
2280
2260
49+00
49+50
50+00
50+50
51+00
STATION



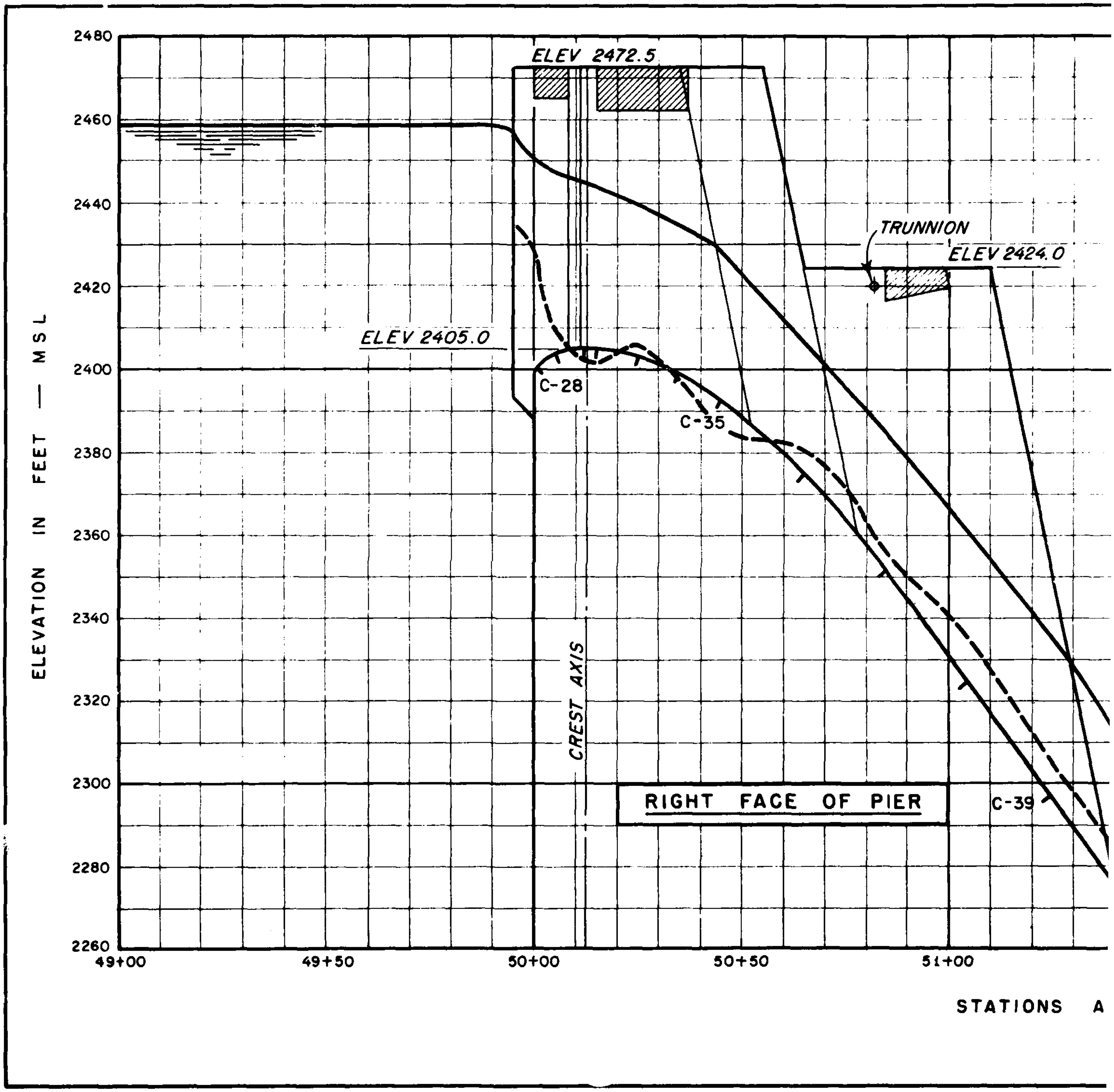
STATIONS ALONG CENTER LINE OF SPILLWAY

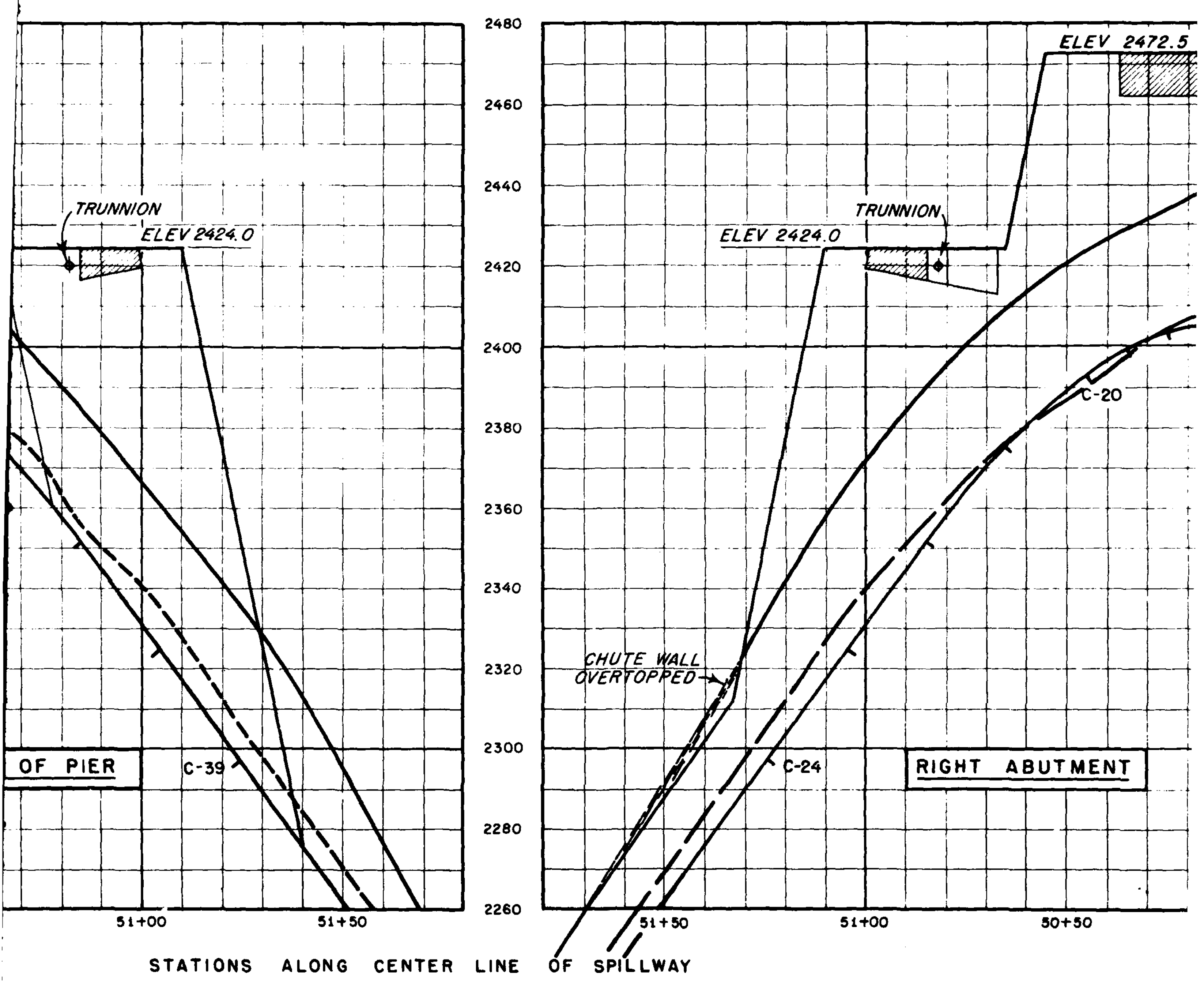
2



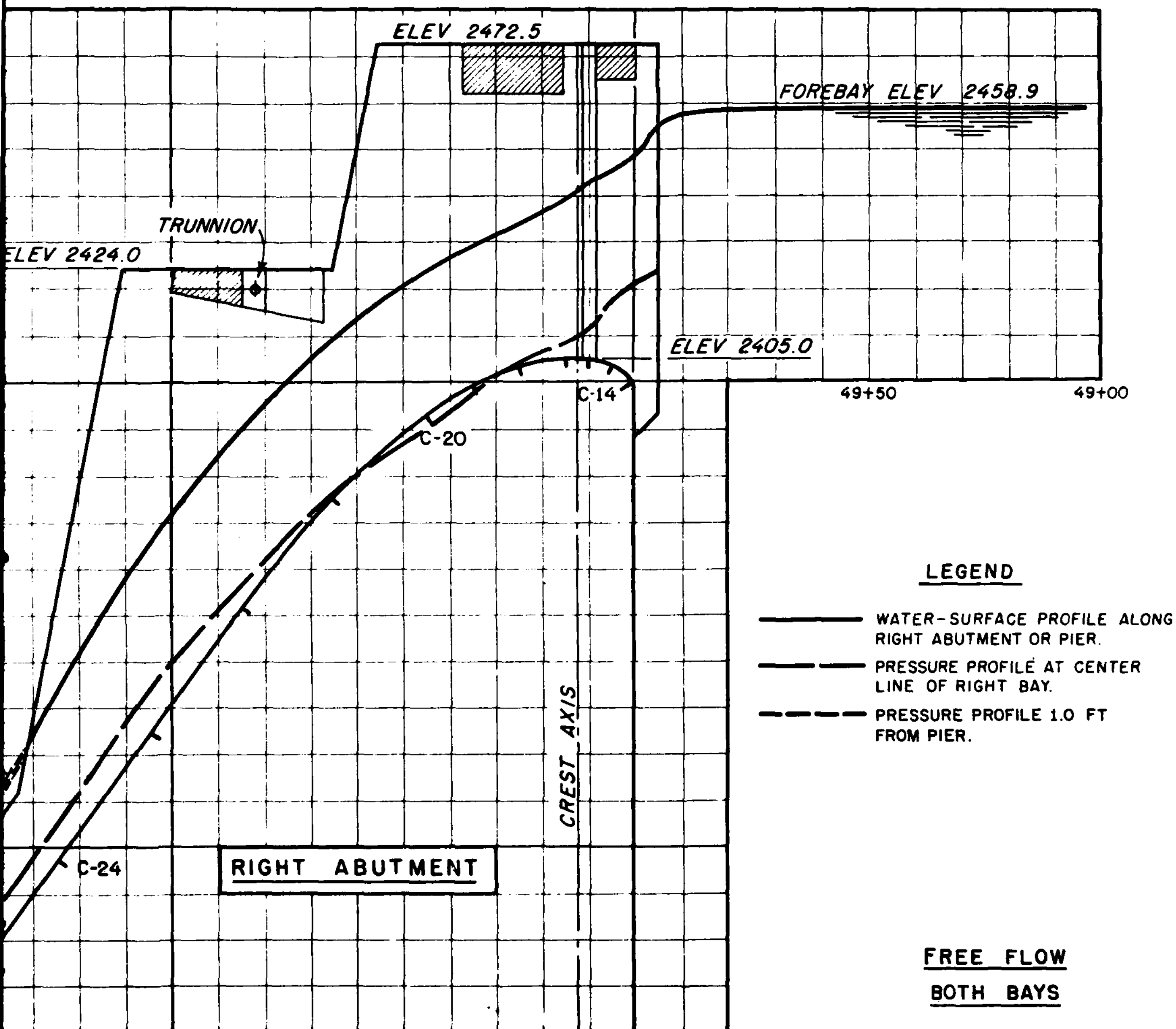
WATER-SURFACE AND PRESSURE PROFILES
PLAN B SPILLWAY STRUCTURES
SPILLWAY DISCHARGE 60 000 CFS

3


ELEV 2472.5
TRUNNION
ELEV 2424.0
ELEV 2405.0
C-28
C-35
C-39
CREST AXIS
RIGHT FACE OF PIER
ELEVATION IN FEET — MSL
2480
2460
2440
2420
2400
2380
2360
2340
2320
2300
2280
2260
49+00
49+50
50+00
50+50
51+00
STATIONS A

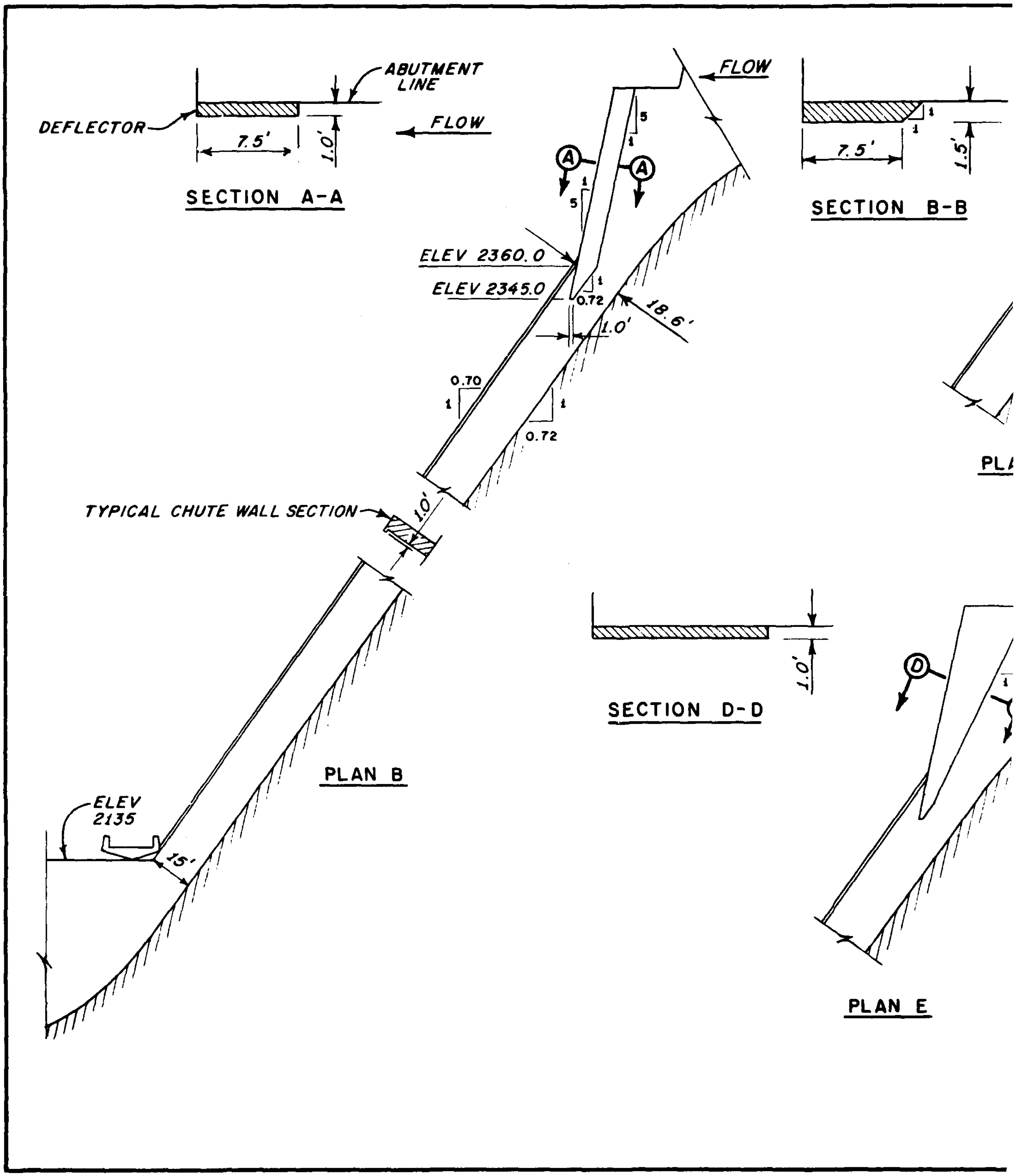

TRUNNION
ELEV 2424.0
OF PIER
C-39
51+00
51+50
ELEV 2472.5
TRUNNION
ELEV 2424.0
C-20
CHUTE WALL
OVERTOPPED
C-24
RIGHT ABUTMENT
51+50
51+00
50+50
2480
2460
2440
2420
2400
2380
2360
2340
2320
2300
2280
2260
STATIONS ALONG CENTER LINE OF SPILLWAY

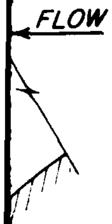
2

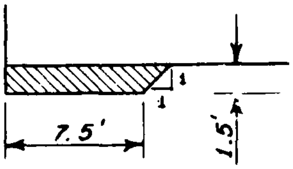


WATER - SURFACE
AND PRESSURE PROFILES
PLAN A SPILLWAY STRUCTURES
SPILLWAY DISCHARGE 145 000 CFS

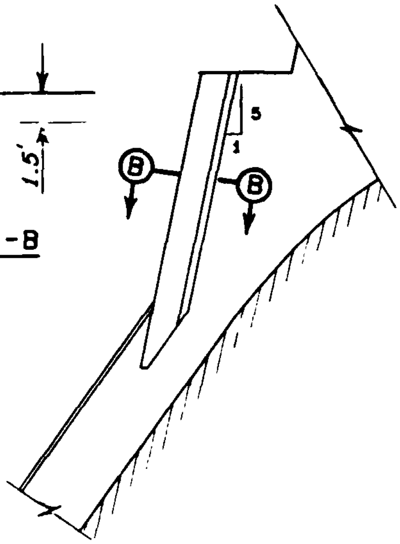
3


ABUTMENT LINE
DEFLECTOR
7.5'
1.0'
FLOW
SECTION A-A
FLOW
5
1
A
A
ELEV 2360.0
ELEV 2345.0
0.72
18.6'
1.0'
0.70
0.72
TYPICAL CHUTE WALL SECTION
1.0'
PLAN B
ELEV 2135
15'
7.5'
1.5'
SECTION B-B
PLA
1.0'
SECTION D-D
D
PLAN E

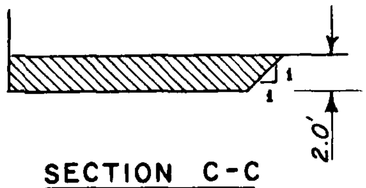




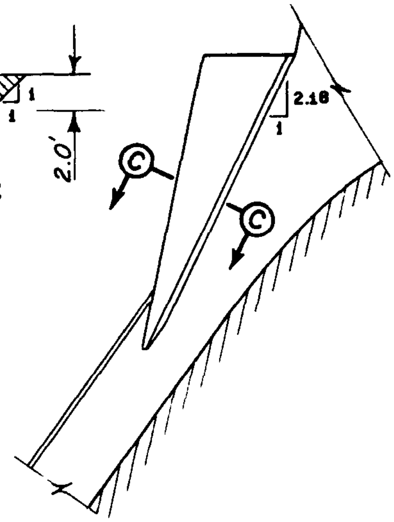
SECTION B-B



PLAN C



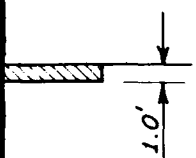
SECTION C-C



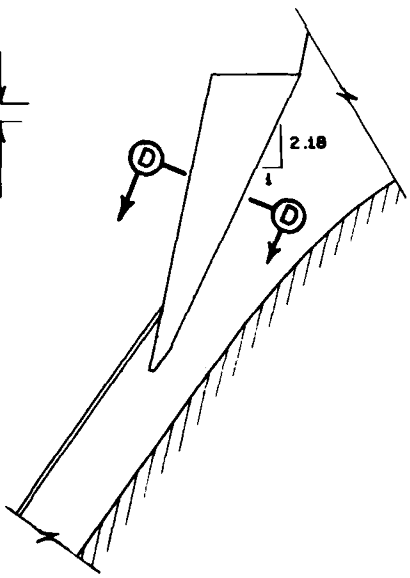
PLAN D

DEFLEC

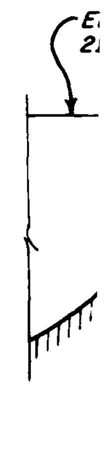
TYPICA



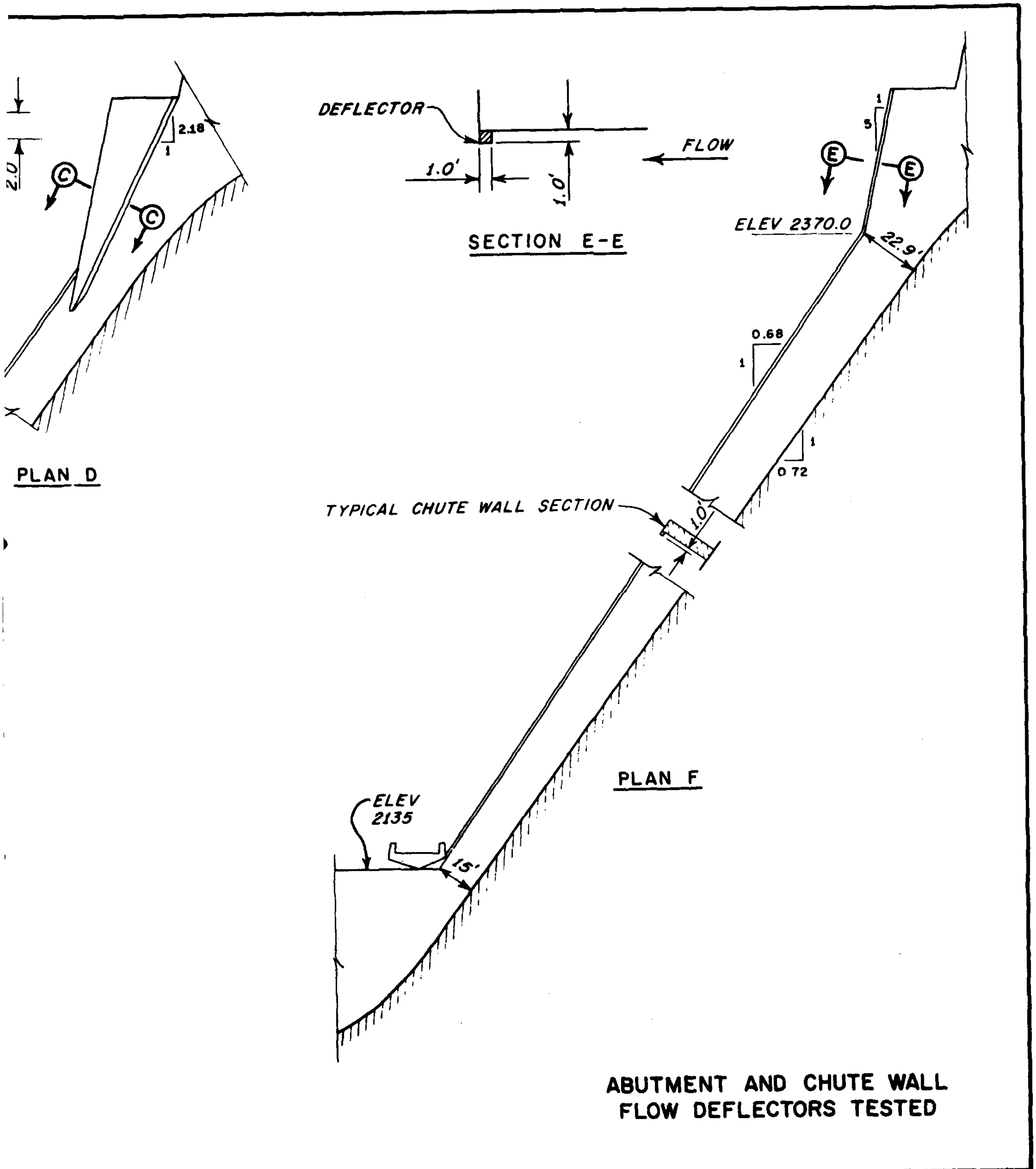
D-D



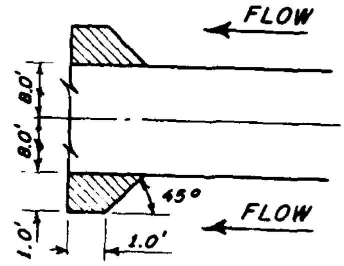
PLAN E



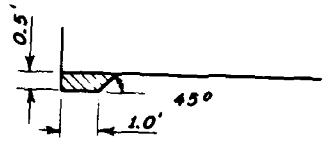
2



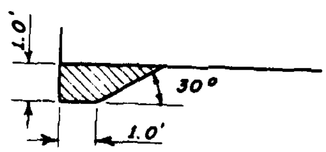
ABUTMENT AND CHUTE WALL
FLOW DEFLECTORS TESTED



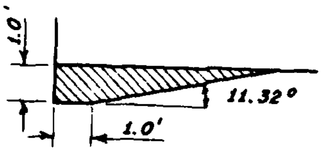
1.0-FT-WIDE, 45-DEGREE



0.5-FT-WIDE, 45-DEGREE

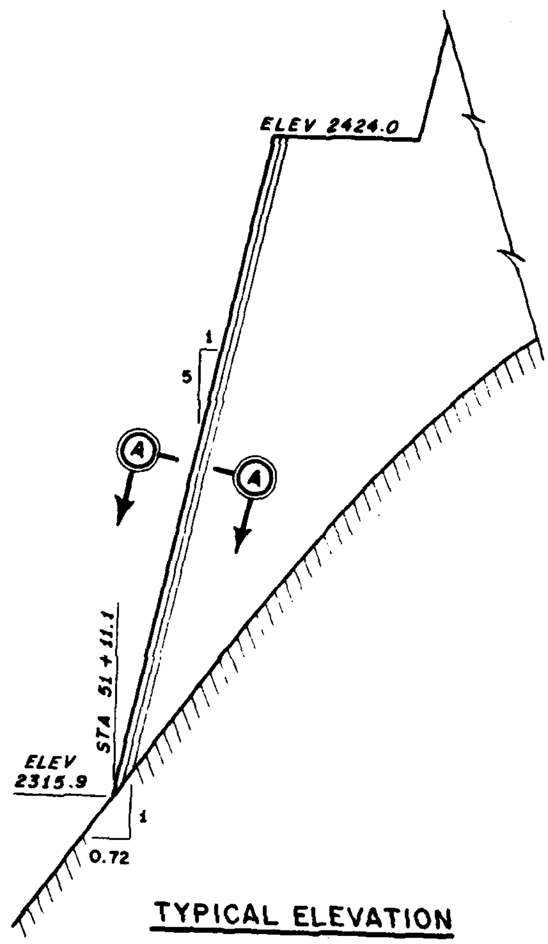


1.0-FT-WIDE, 30-DEGREE



1.0-FT-WIDE, 11.32-DEGREE

SECTIONS A-A

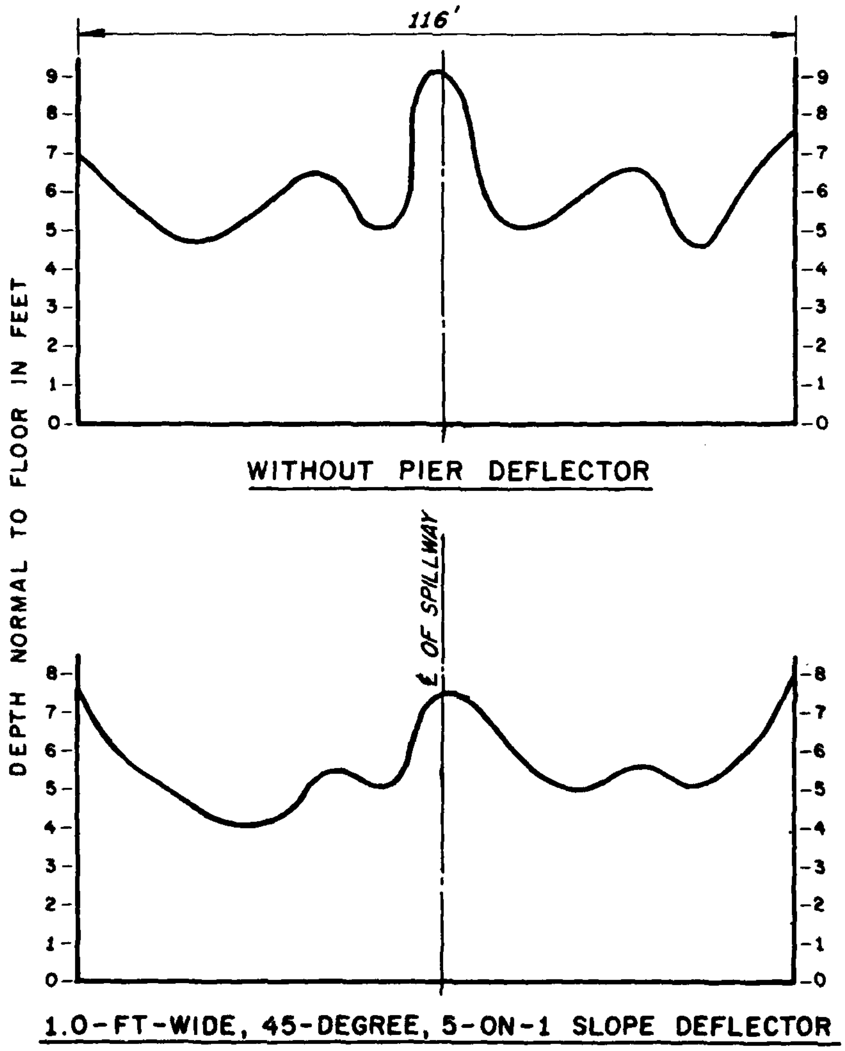


TYPICAL ELEVATION

NOTE

DETAILS OF PLAN B PIER ARE SHOWN ON PLATE 16.

FLOW DEFLECTORS TESTED ON PLAN B SPILLWAY PIER

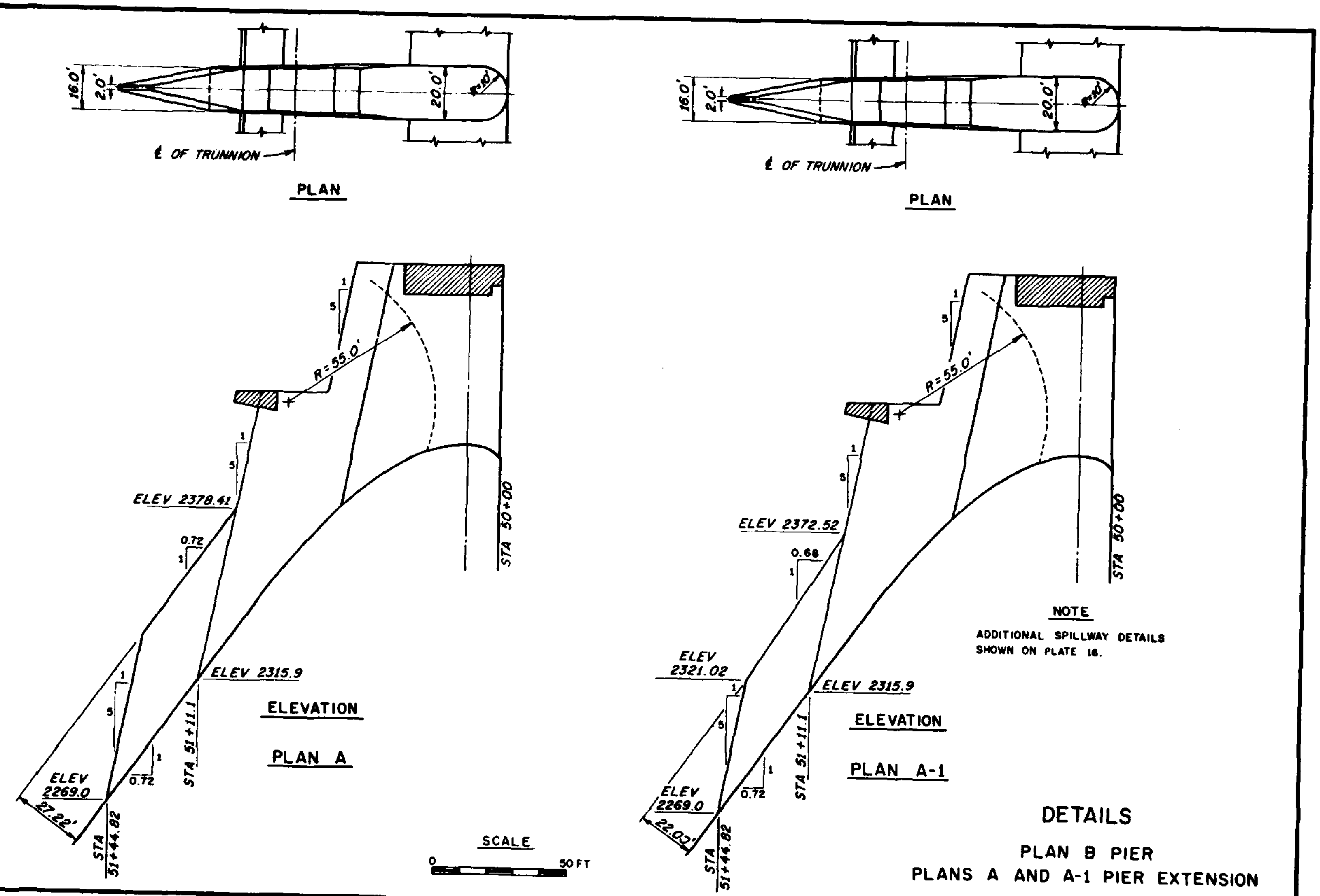


NOTES

1. DETAILS OF STILLING BASIN, PIER AND PIER DEFLECTORS SHOWN ON PLATES 16, 17 AND 22.
2. PROFILES TAKEN UPSTREAM OF HYDRAULIC JUMP.
3. FOREBAY ELEV 2459.

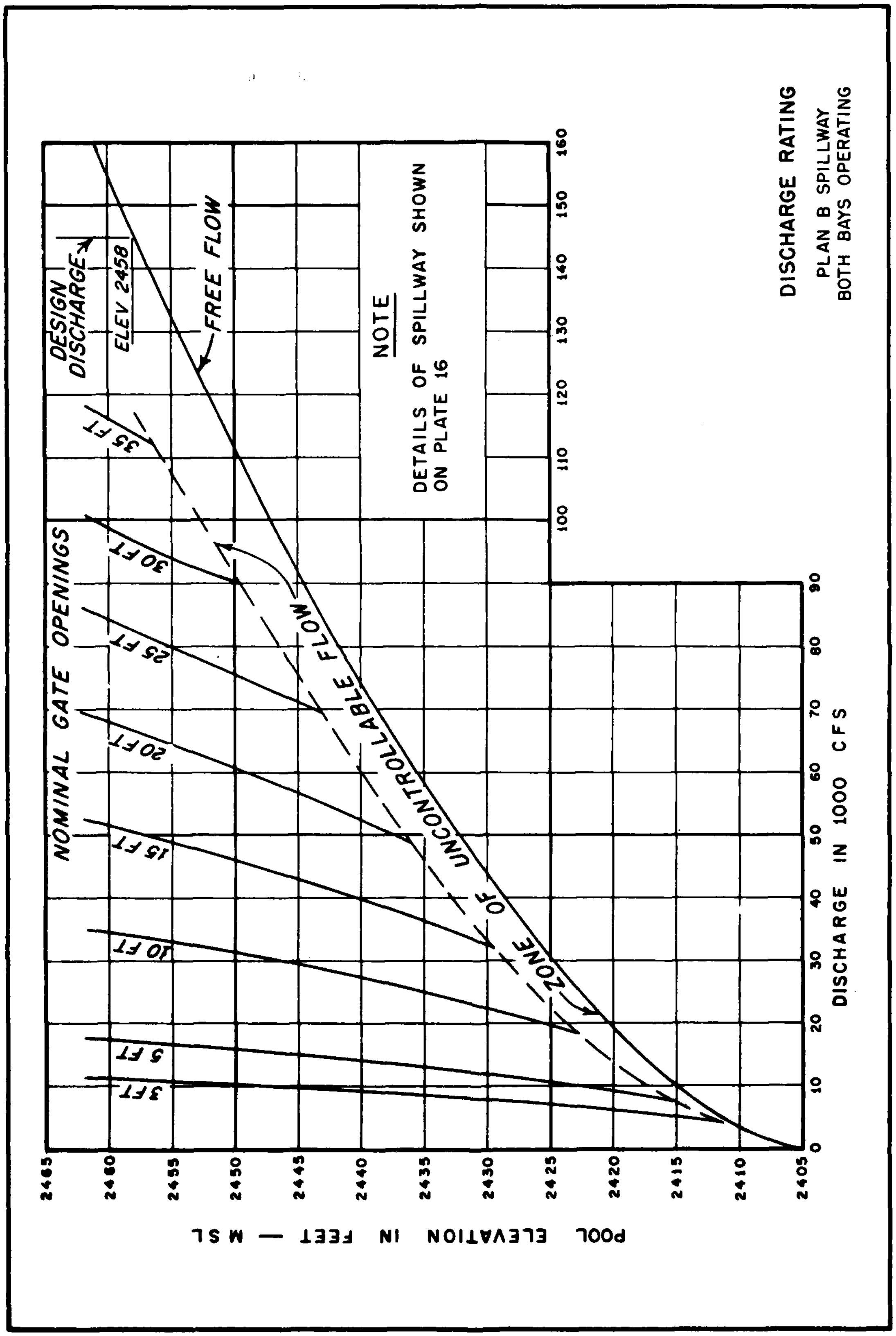
FLOW PROFILES AT STATION 54+00

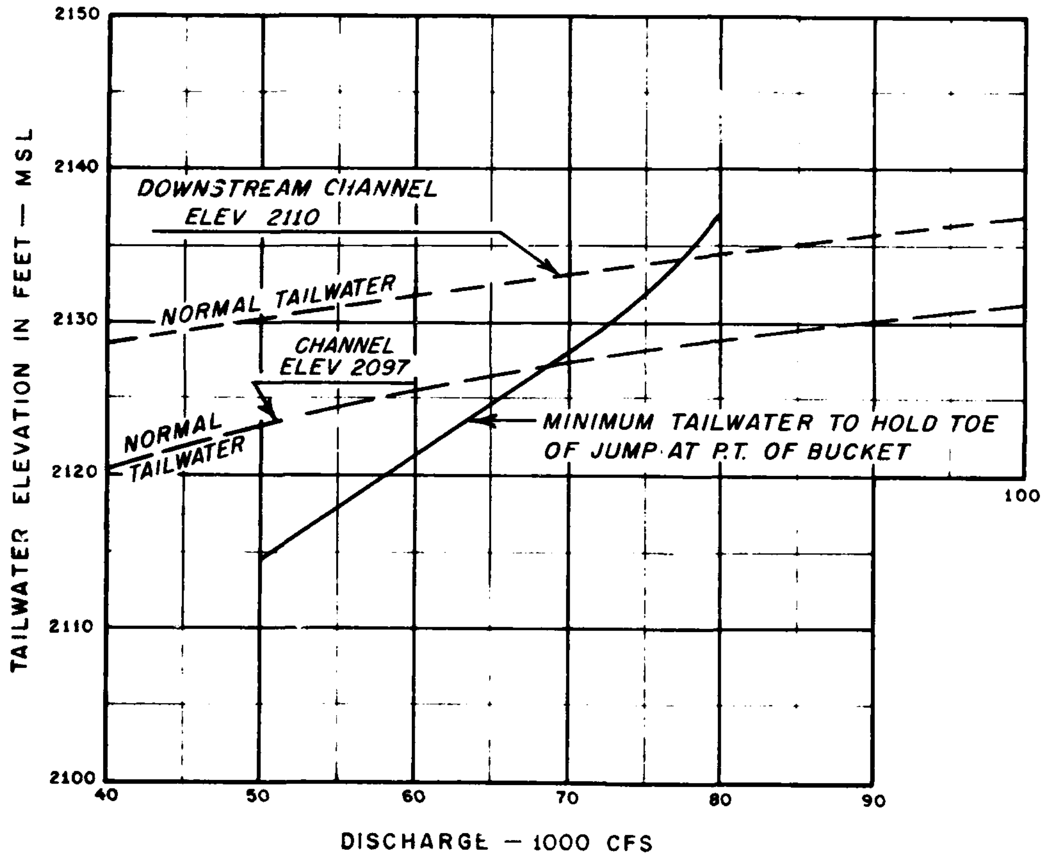
PLAN C STILLING BASIN
PLAN B SPILLWAY STRUCTURES



DETAILS

PLAN B PIER
PLANS A AND A-1 PIER EXTENSION


DISCHARGE RATING
PLAN B SPILLWAY
BOTH BAYS OPERATING
NOTE
DETAILS OF SPILLWAY SHOWN
ON PLATE 16
DESIGN
DISCHARGE
ELEV 2458
FREE FLOW
NOMINAL GATE OPENINGS
35 FT
30 FT
25 FT
20 FT
15 FT
10 FT
5 FT
3 FT
ZONE OF UNCONTROLLABLE FLOW
DISCHARGE IN 1000 CFS
POOL ELEVATION IN FEET — M S L
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
2405
2410
2415
2420
2425
2430
2435
2440
2445
2450
2455
2460
2465

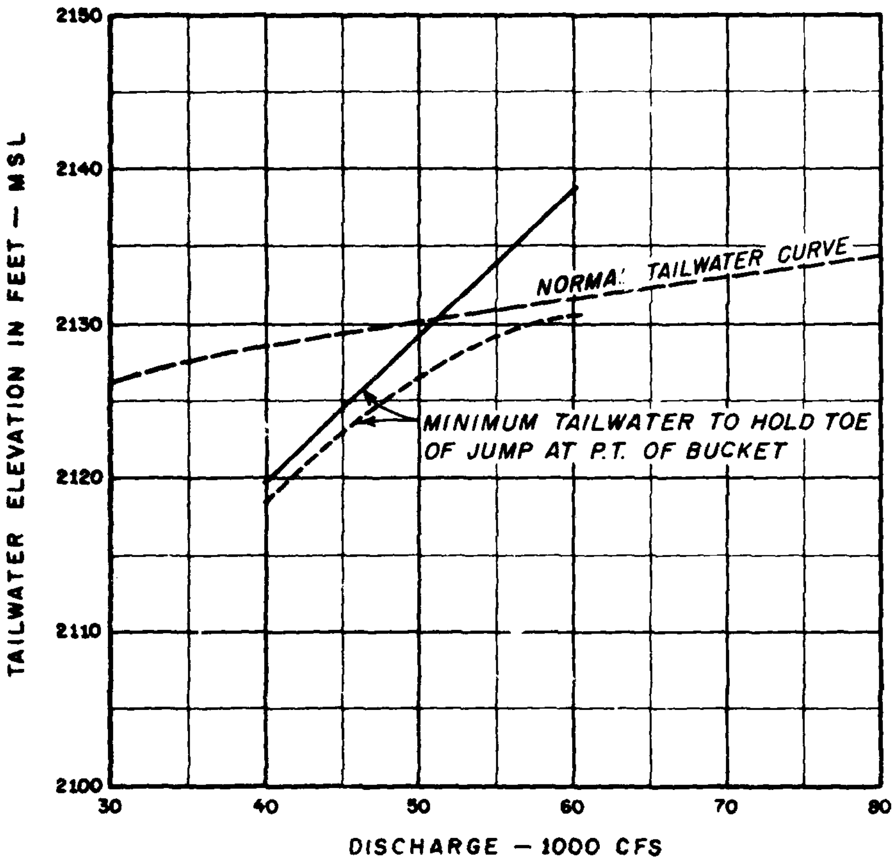


NOTES

1. DETAILS OF STILLING BASIN SHOWN ON PLATE 17.
2. DETAILS OF CHANNEL AND LOCATION OF TAILWATER GAGE SHOWN ON PLATES 4 AND 28.
3. FOREBAY ELEV 2459.

MINIMUM TAILWATER
FOR HYDRAULIC JUMP

PLAN C STILLING BASIN
CHANNEL ELEVATIONS 2110 AND 2097



NOTES

1. DETAILS OF STILLING BASIN SHOWN ON PLATE 29.
2. DETAILS OF CHANNEL AND LOCATION OF TAILWATER GAGE SHOWN ON PLATE 28.
3. FOREBAY ELEV 2459.

LEGEND

——— SPILLWAY FLOW

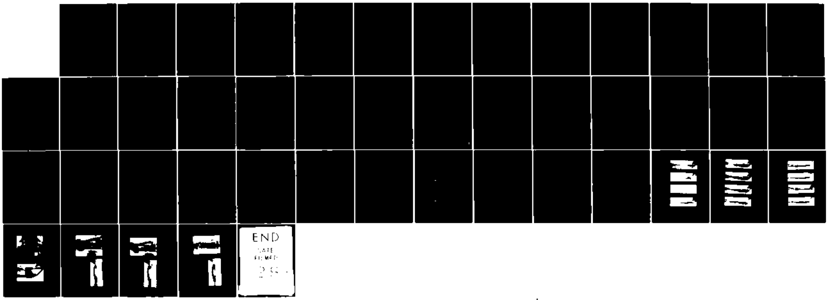
- - - - SLUICE FLOW

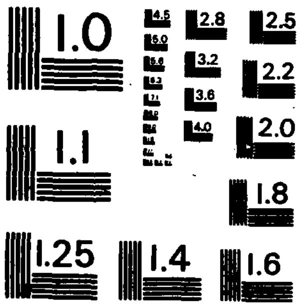
MINIMUM TAILWATER FOR HYDRAULIC JUMP

PLAN D STILLING BASIN
CHANNEL ELEVATION 2110

AD-A134 655 HYDRAULIC MODEL INVESTIGATION LIBBY DAM KOOTENAI RIVER MONTANA HYDRAULIC..(U) ARMY ENGINEER DIV NORTH PACIFIC BONNEVILLE OR DIV HYDRAULIC L.. L Z PERKINS ET AL. 2/2

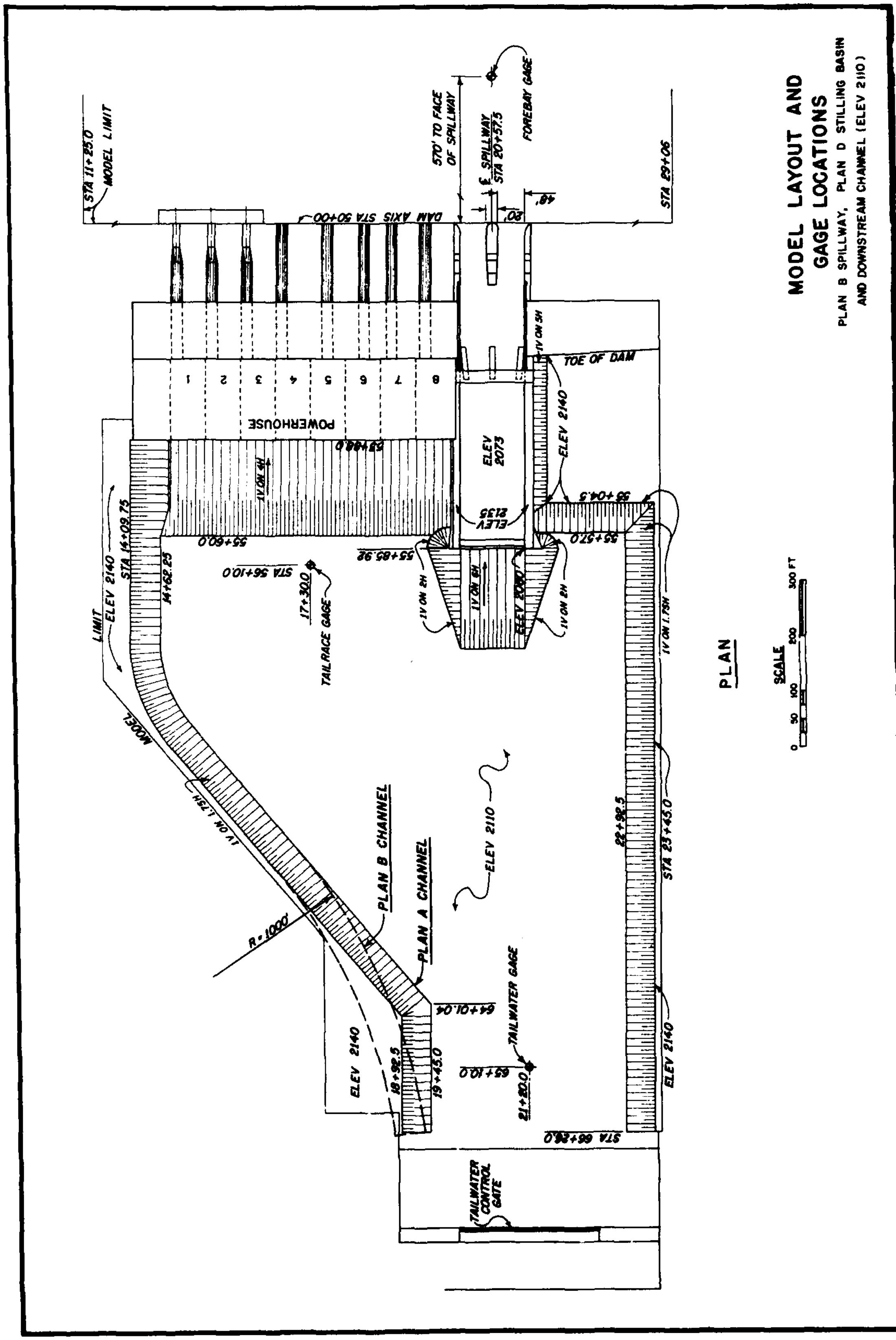
UNCLASSIFIED JUN 83 TR-125-1 F/G 13/2 NL





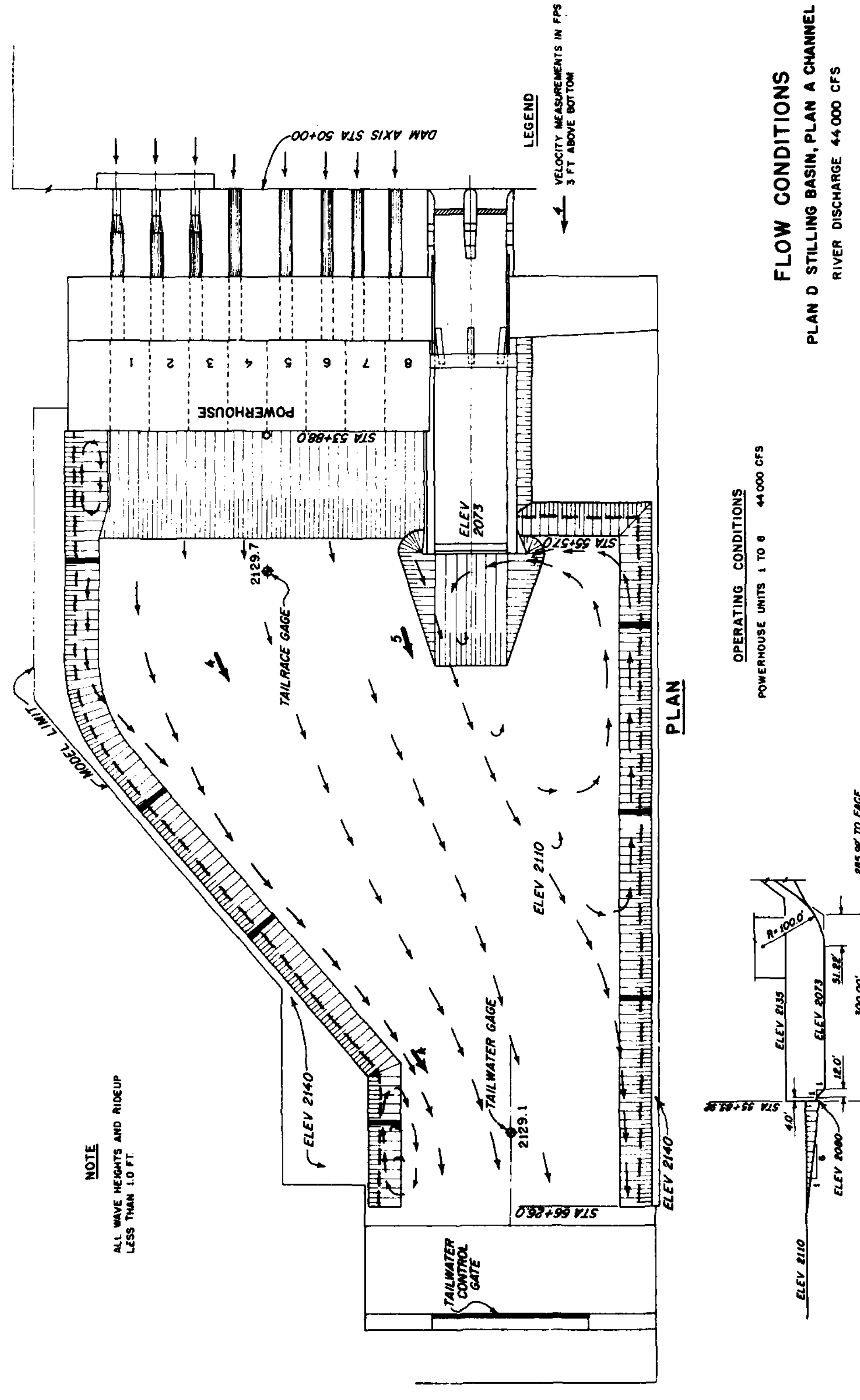
MICROCOPY RESOLUTION TEST CHART

NATIONAL BUREAU OF STANDARDS-1963-A

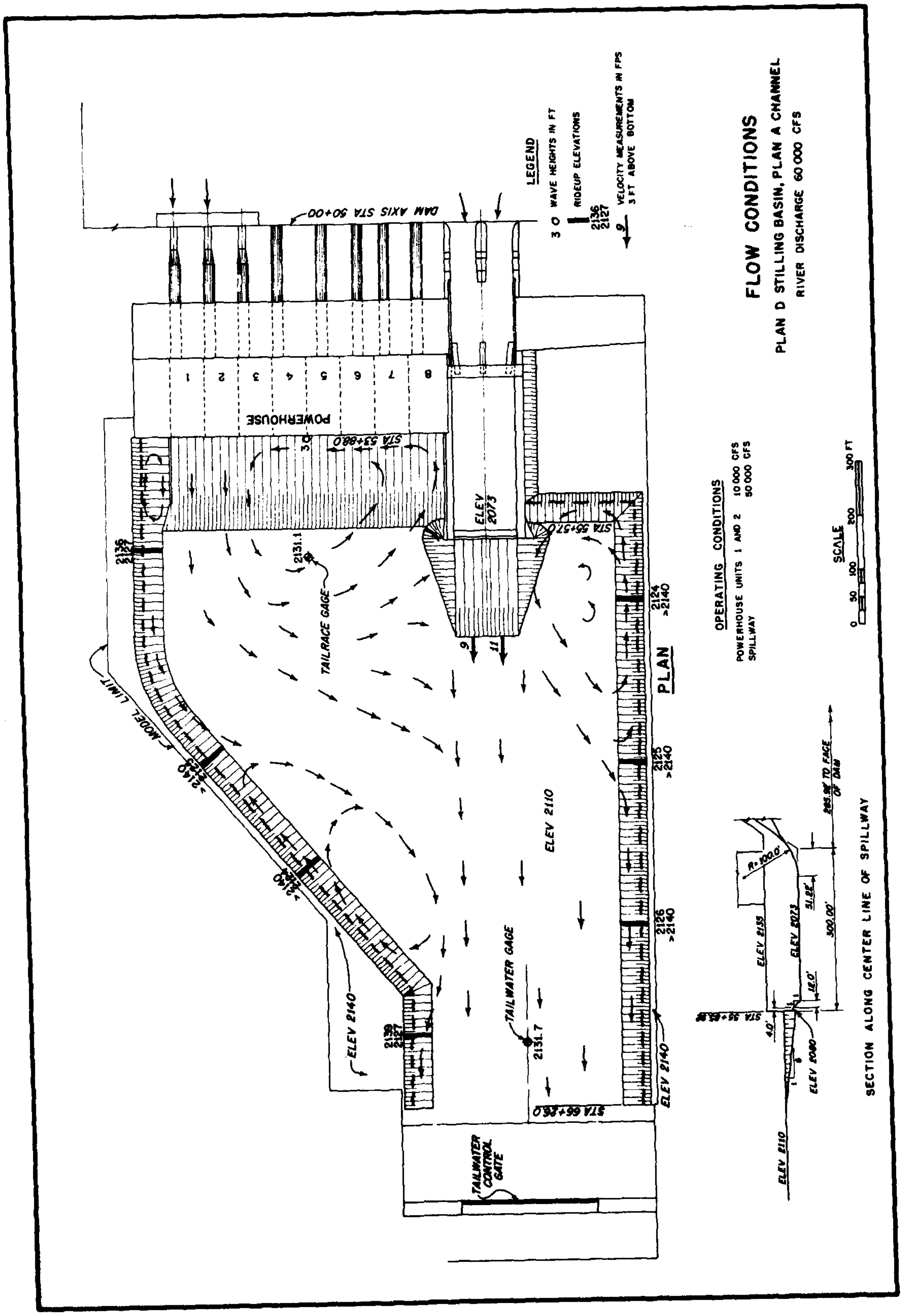


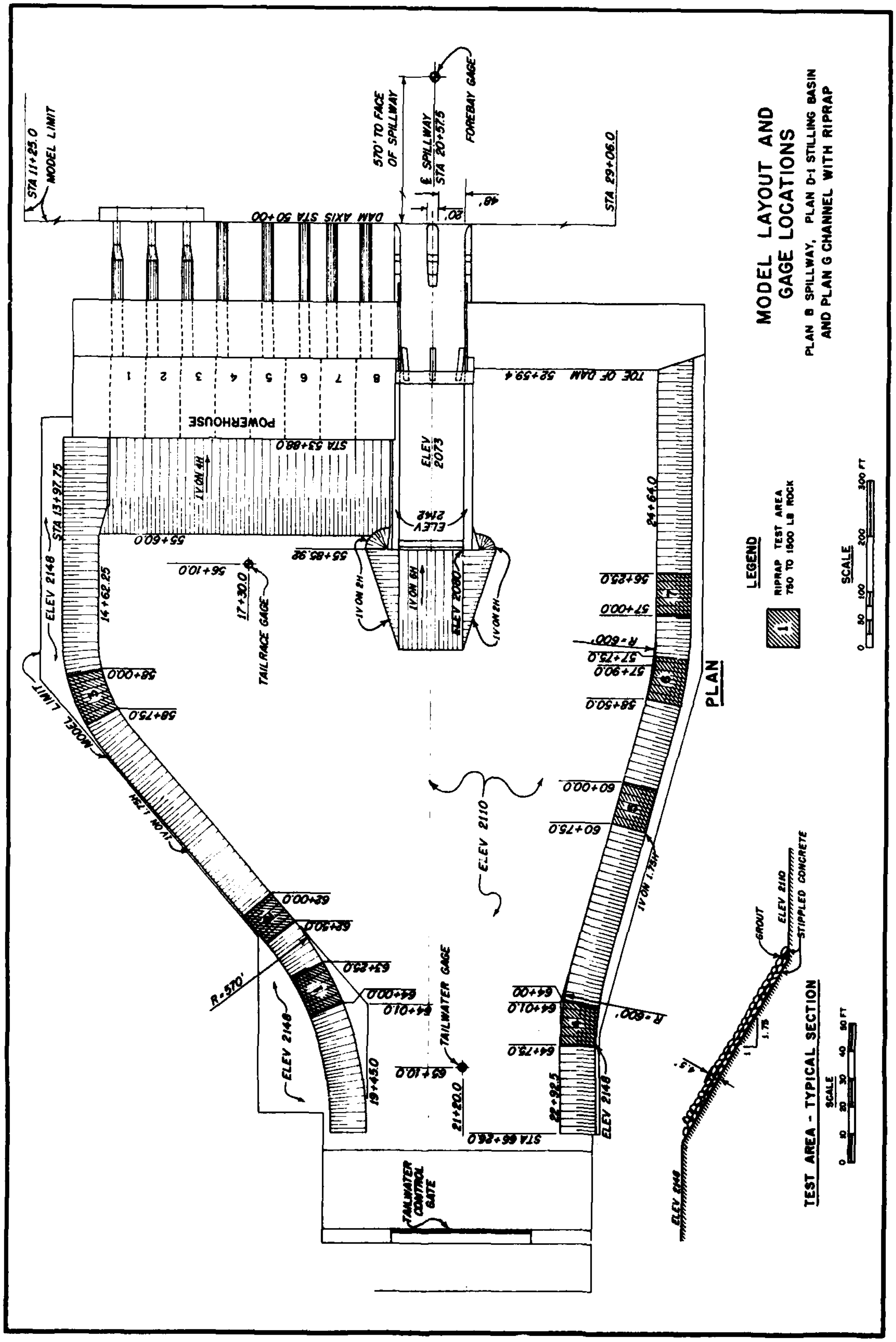
MODEL LAYOUT AND GAGE LOCATIONS

PLAN B SPILLWAY, PLAN D STILLING BASIN AND DOWNSTREAM CHANNEL (ELEV 2110)



FLOW CONDITIONS
PLAN D STILLING BASIN, PLAN A CHANNEL
RIVER DISCHARGE 44000 CFS


FLOW CONDITIONS
PLAN D STILLING BASIN, PLAN A CHANNEL
RIVER DISCHARGE 60 000 CFS
LEGEND
3.0 WAVE HEIGHTS IN FT
RIDEUP ELEVATIONS
2136
2127
VELOCITY MEASUREMENTS IN FPS 3 FT ABOVE BOTTOM
OPERATING CONDITIONS
POWERHOUSE UNITS 1 AND 2 10 000 CFS
SPILLWAY 50 000 CFS
SCALE
0 50 100 200 300 FT
DAM AXIS STA 50+00
POWERHOUSE
1 2 3 4 5 6 7 8
STA 53+88.0
ELEV 2073
STA 55+57.0
TAILRACE GAGE
2131.1
MODEL LIMIT
ELEV 2110
TAILWATER GAGE
2131.7
ELEV 2140
STA 66+26.0
TAILWATER CONTROL GATE
PLAN
R=100.0'
ELEV 2135
ELEV 2073
300.00'
18.0'
4.0'
ELEV 2080
ELEV 2110
SECTION ALONG CENTER LINE OF SPILLWAY


MODEL LAYOUT AND GAGE LOCATIONS
PLAN B SPILLWAY, PLAN D-1 STILLING BASIN AND PLAN G CHANNEL WITH RIPRAP
PLAN
LEGEND
RIPRAP TEST AREA 750 TO 1500 LB ROCK
SCALE
TEST AREA - TYPICAL SECTION
POWERHOUSE
FOREBAY GAGE
TAILRACE GAGE
TAILWATER GAGE
TAILWATER CONTROL GATE
DAM AXIS STA 50+00
TOE OF DAM 52+59.4
STA 11+25.0 MODEL LIMIT
STA 29+06.0
570' TO FACE OF SPILLWAY
℄ SPILLWAY STA 20+57.5
ELEV 2073
ELEV 2110
ELEV 2148
GROUT
STIPPLED CONCRETE
R=570'
R=600'

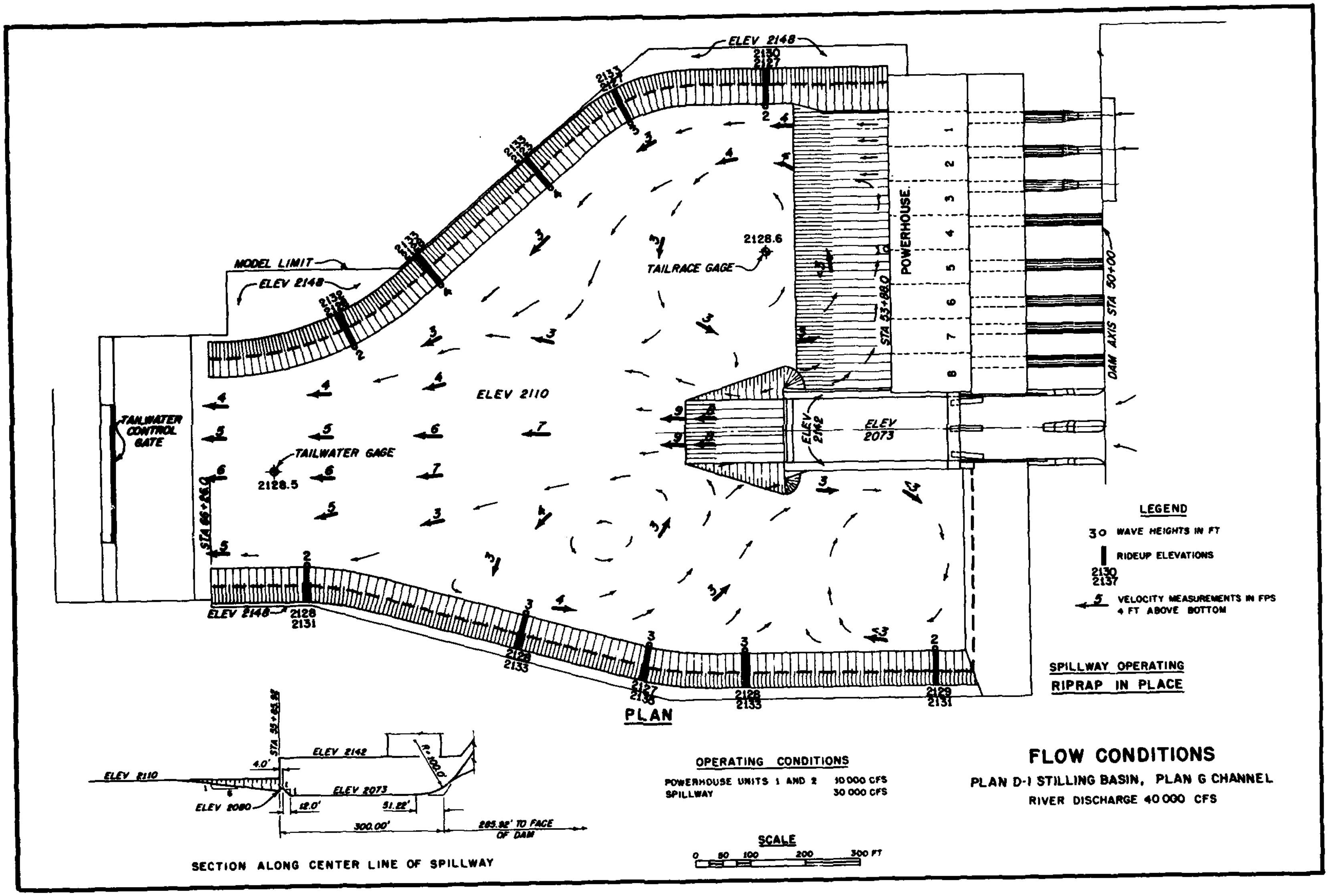
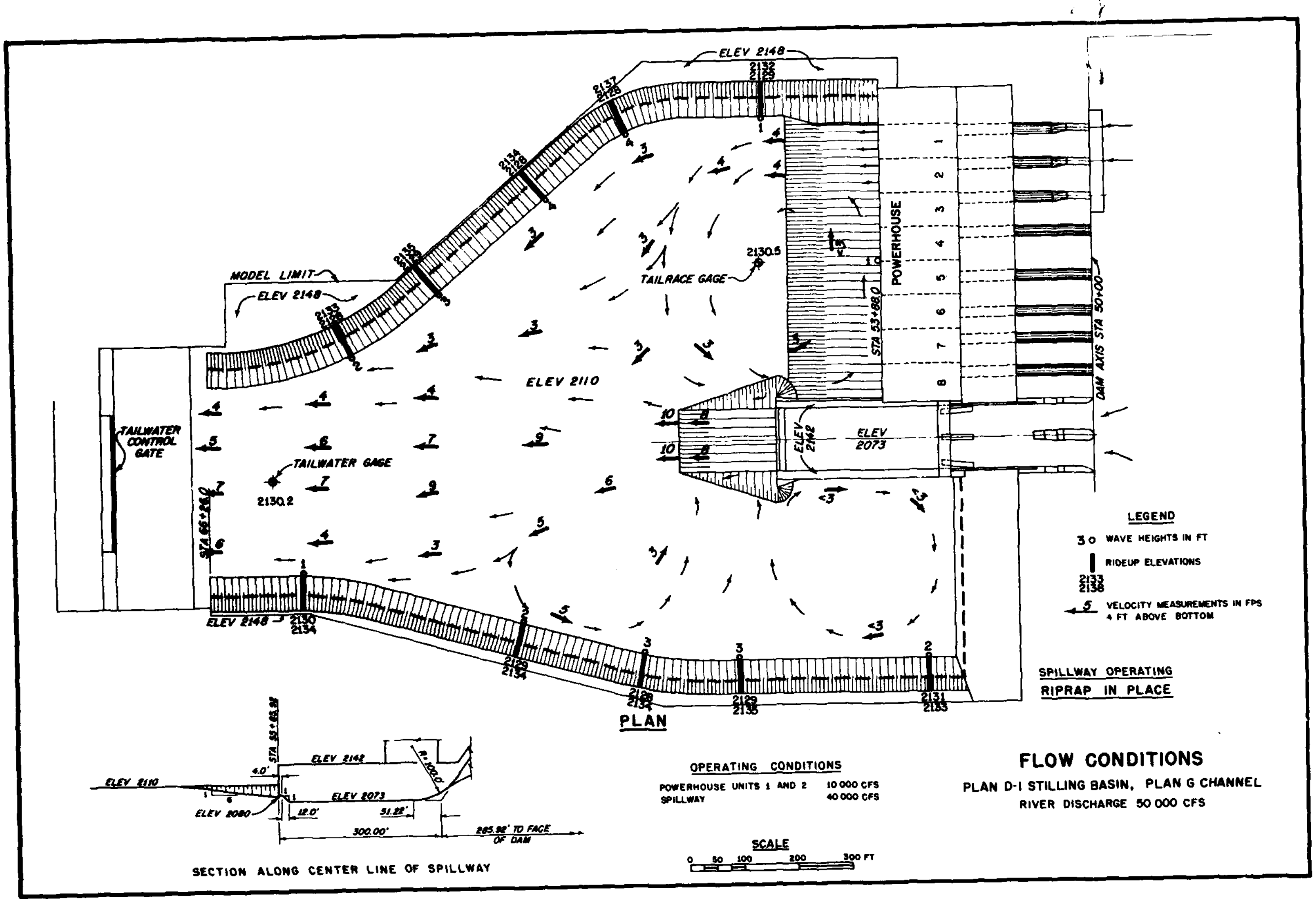

FLOW CONDITIONS
PLAN D-1 STILLING BASIN, PLAN G CHANNEL
RIVER DISCHARGE 40 000 CFS
LEGEND
3○ WAVE HEIGHTS IN FT
RIDEUP ELEVATIONS
2130
2137
5 VELOCITY MEASUREMENTS IN FPS 4 FT ABOVE BOTTOM
SPILLWAY OPERATING
RIPRAP IN PLACE
OPERATING CONDITIONS
POWERHOUSE UNITS 1 AND 2 10 000 CFS
SPILLWAY 30 000 CFS
SCALE
0 50 100 200 300 FT
PLAN
POWERHOUSE
DAM AXIS STA 50+00
STA 53+88.0
STA 66+88.0
ELEV 2148
ELEV 2110
ELEV 2073
ELEV 2142
MODEL LIMIT
TAILRACE GAGE
2128.6
TAILWATER GAGE
2128.5
TAILWATER CONTROL GATE
SECTION ALONG CENTER LINE OF SPILLWAY
ELEV 2110
ELEV 2080
ELEV 2142
ELEV 2073
STA 55+65.92
R=100.0'
4.0'
12.0'
51.22'
300.00'
285.92' TO FACE OF DAM

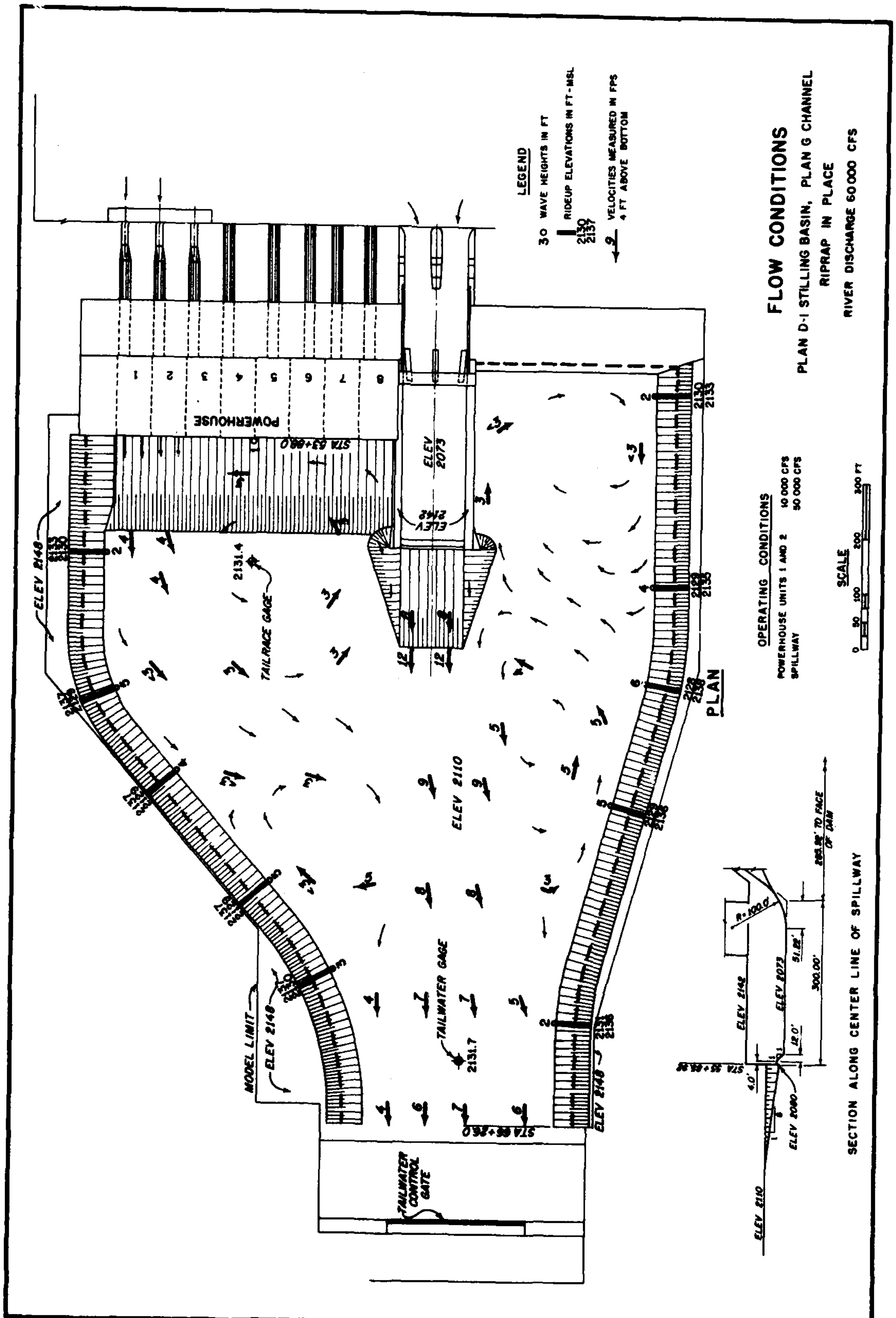
PLATE 32

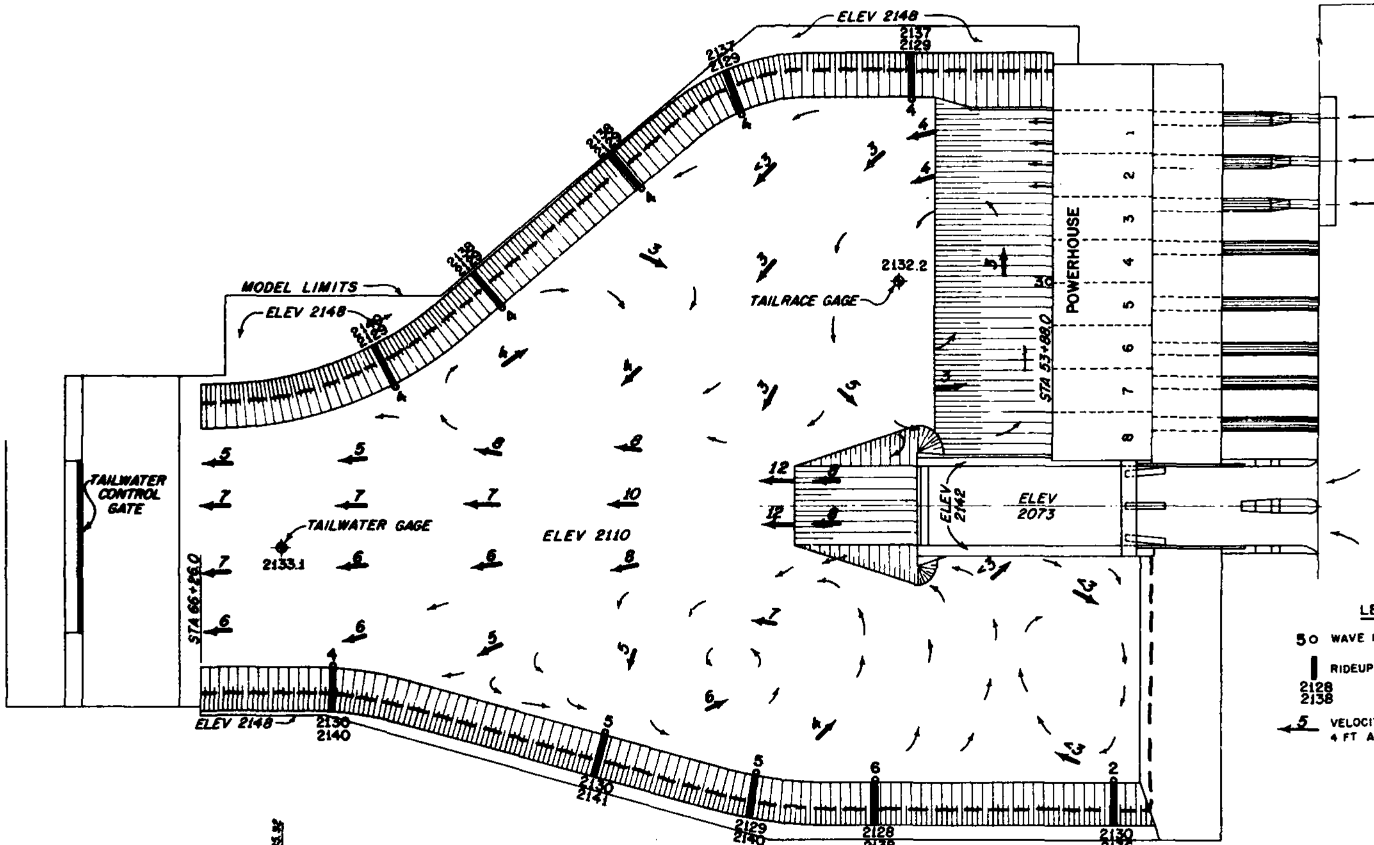


FLOW CONDITIONS

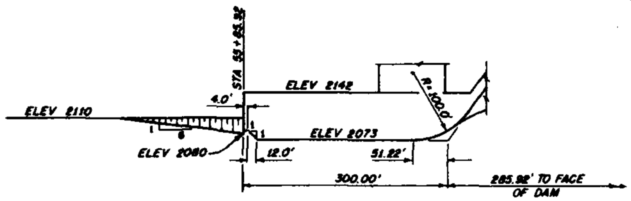
PLAN D-1 STILLING BASIN, PLAN G CHANNEL

RIVER DISCHARGE 50 000 CFS


FLOW CONDITIONS
PLAN D-1 STILLING BASIN, PLAN G CHANNEL
RIPRAP IN PLACE
RIVER DISCHARGE 60 000 CFS
LEGEND
3.0 WAVE HEIGHTS IN FT
RIDEUP ELEVATIONS IN FT - MSL
VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
POWERHOUSE UNITS 1 AND 2 10 000 CFS
SPILLWAY 50 000 CFS
SCALE
PLAN
POWERHOUSE
TAILRACE GAGE
TAILWATER GAGE
TAILWATER CONTROL GATE
MODEL LIMIT
ELEV 2148
ELEV 2110
ELEV 2073
ELEV 2142
SECTION ALONG CENTER LINE OF SPILLWAY



PLAN



SECTION ALONG CENTER LINE OF SPILLWAY

LEGEND

5 ○ WAVE HEIGHTS IN FT

2128 / 2138 RIDEUP ELEVATIONS IN FT-MSL

5 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM

OPERATING CONDITIONS

POWERHOUSE UNITS 1 AND 2	10 000 CFS
SPILLWAY	60 000 CFS

SCALE

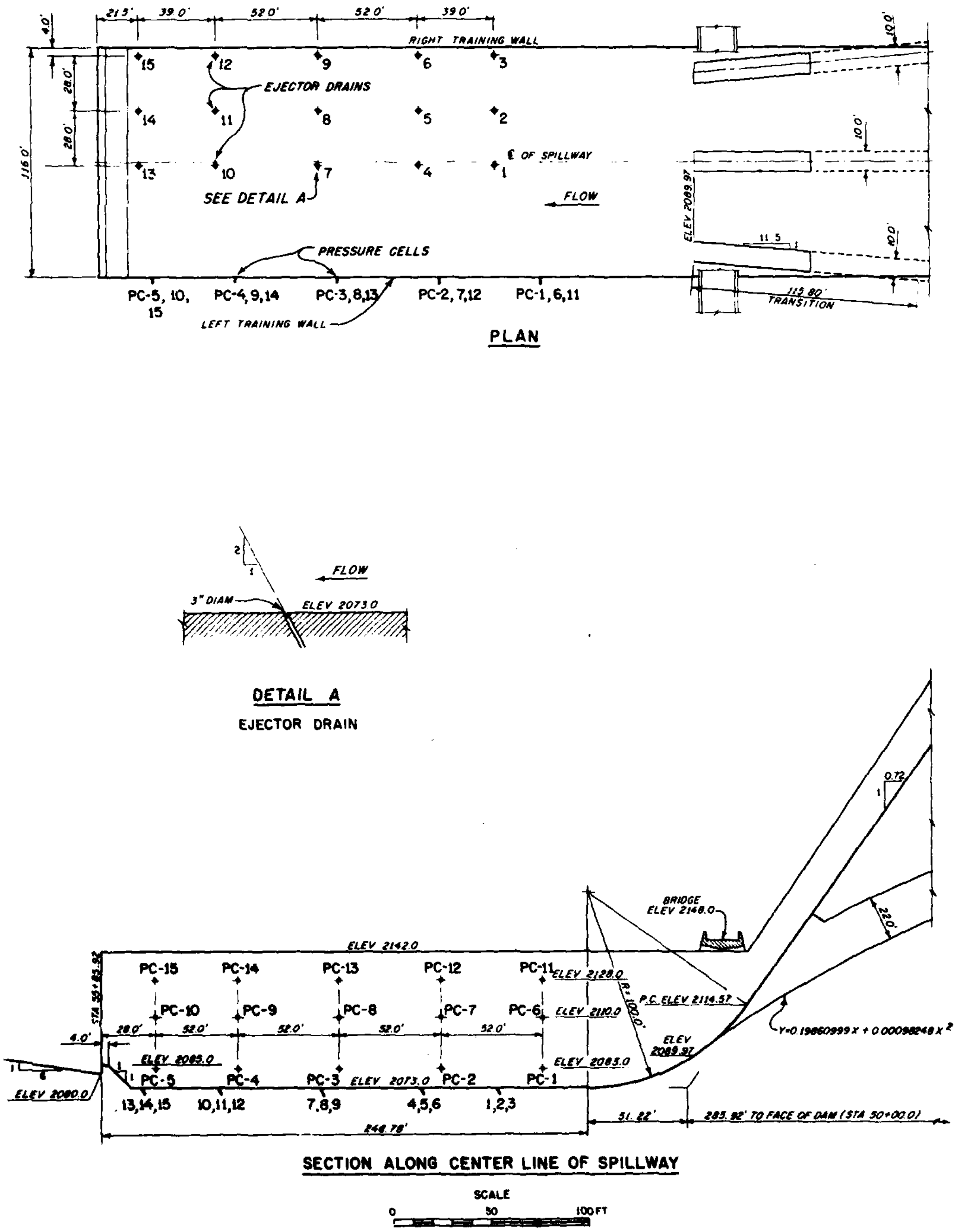
0 50 100 200 300 FT

FLOW CONDITIONS

PLAN D-1 STILLING BASIN, PLAN G CHANNEL

RIPRAP IN PLACE

RIVER DISCHARGE 70 000 CFS



NOTE

PRESSURE CELLS SHOWN IN SECTIONAL VIEW ARE LOCATED IN LEFT WALL

LOCATION OF EJECTOR DRAINS AND PRESSURE CELLS

PLAN D-1 STILLING BASIN

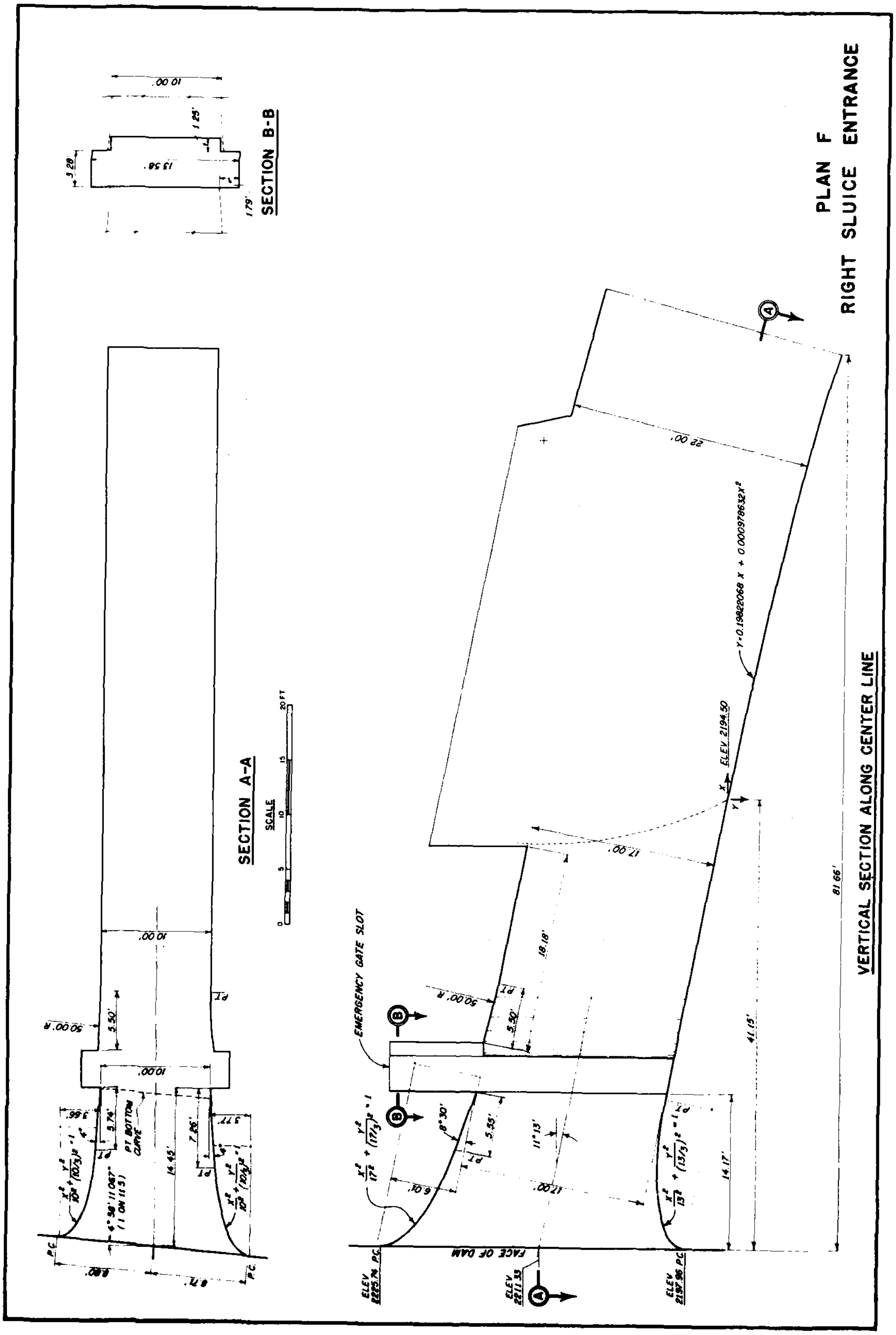
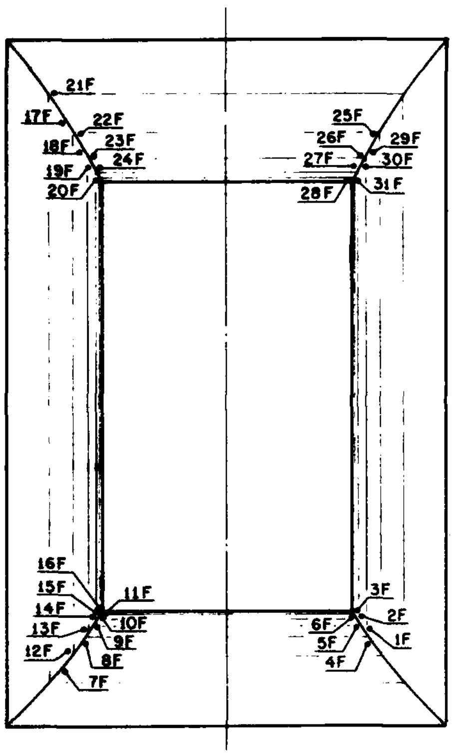
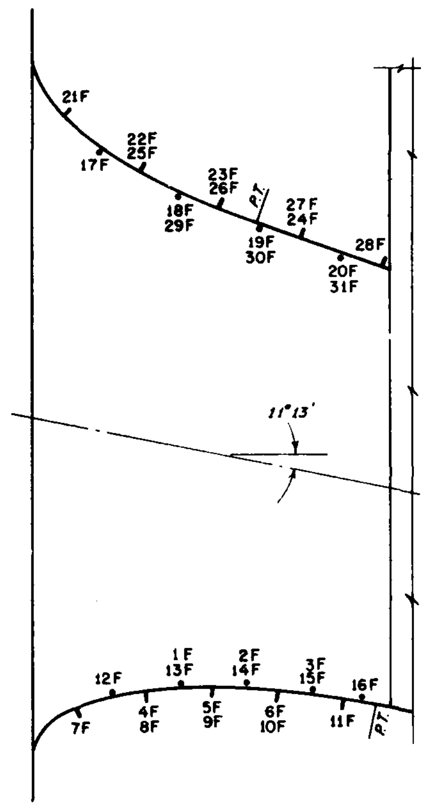

PLAN F
RIGHT SLUICE ENTRANCE
VERTICAL SECTION ALONG CENTER LINE
SECTION A-A
SECTION B-B
SCALE
EMERGENCY GATE SLOT
FACE OF DAM
ELEV 2225.74 P.C.
ELEV 2211.33
ELEV 2197.96 P.C.
ELEV 2194.50
Y=0.19822068 X + 0.000978632X²
81.66'
41.15'
22.00'
17.00'
18.18'
12.00'
14.17'
10.00'
50.00' R
5.50'

PLATE 37



FACE VIEW NORMAL TO CENTER LINE

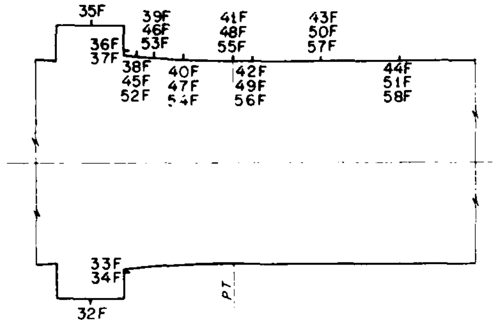


SECTION ALONG CE..ER LINE

PIEZOMETER LOCATIONS					
PIEZ NO.	STATION	ELEV	PIEZ NO.	STATION	ELEV
1F	1+05.3	2200.9	17F	1+03.2	2221.9
2F	1+08.0	2200.9	18F	1+06.4	2220.1
3F	1+10.6	2200.6	19F	1+09.5	2218.8
			20F	1+12.7	2217.6
4F	1+04.0	2200.6			
5F	1+06.7	2200.7	21F	1+01.8	2223.3
6F	1+09.3	2200.6	22F	1+04.8	2221.1
			23F	1+07.9	2219.6
7F	1+02.3	2199.7	24F	1+11.1	2218.4
8F	1+05.0	2200.4			
9F	1+07.6	2200.5	25F	1+03.8	2221.3
10F	1+10.2	2200.4	26F	1+07.0	2219.8
11F	1+12.8	2200.1	27F	1+10.2	2218.6
			28F	1+13.5	2217.5
12F	1+03.6	2200.3			
13F	1+06.3	2200.7	29F	1+05.4	2220.3
14F	1+08.9	2200.7	30F	1+08.6	2219.0
15F	1+11.5	2200.4	31F	1+11.8	2217.8
16F	1+13.4	2200.2			

NOTE
ENTRANCE DETAILS SHOWN ON PLATE 37.

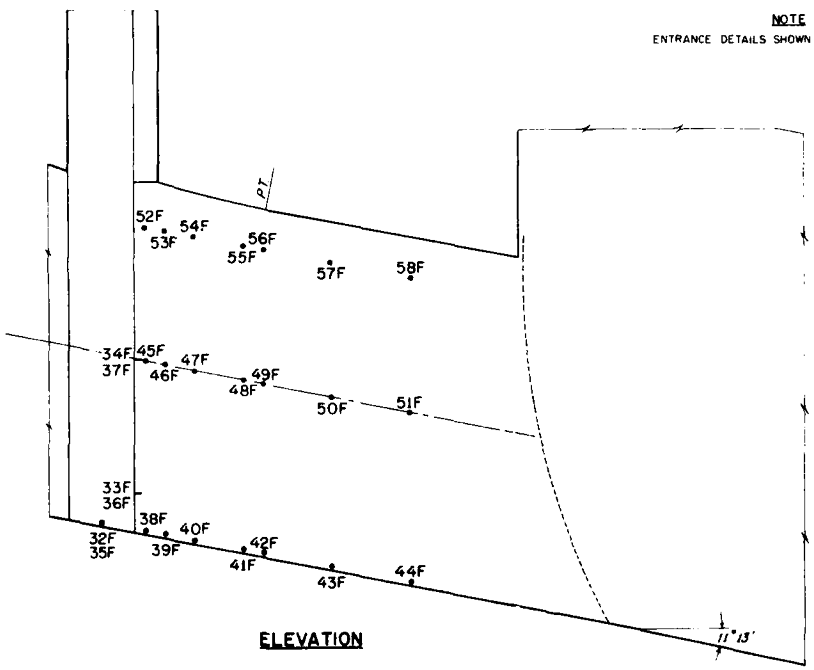
PIEZOMETER LOCATIONS
PLAN F RIGHT SLUICE ENTRANCE



PLAN

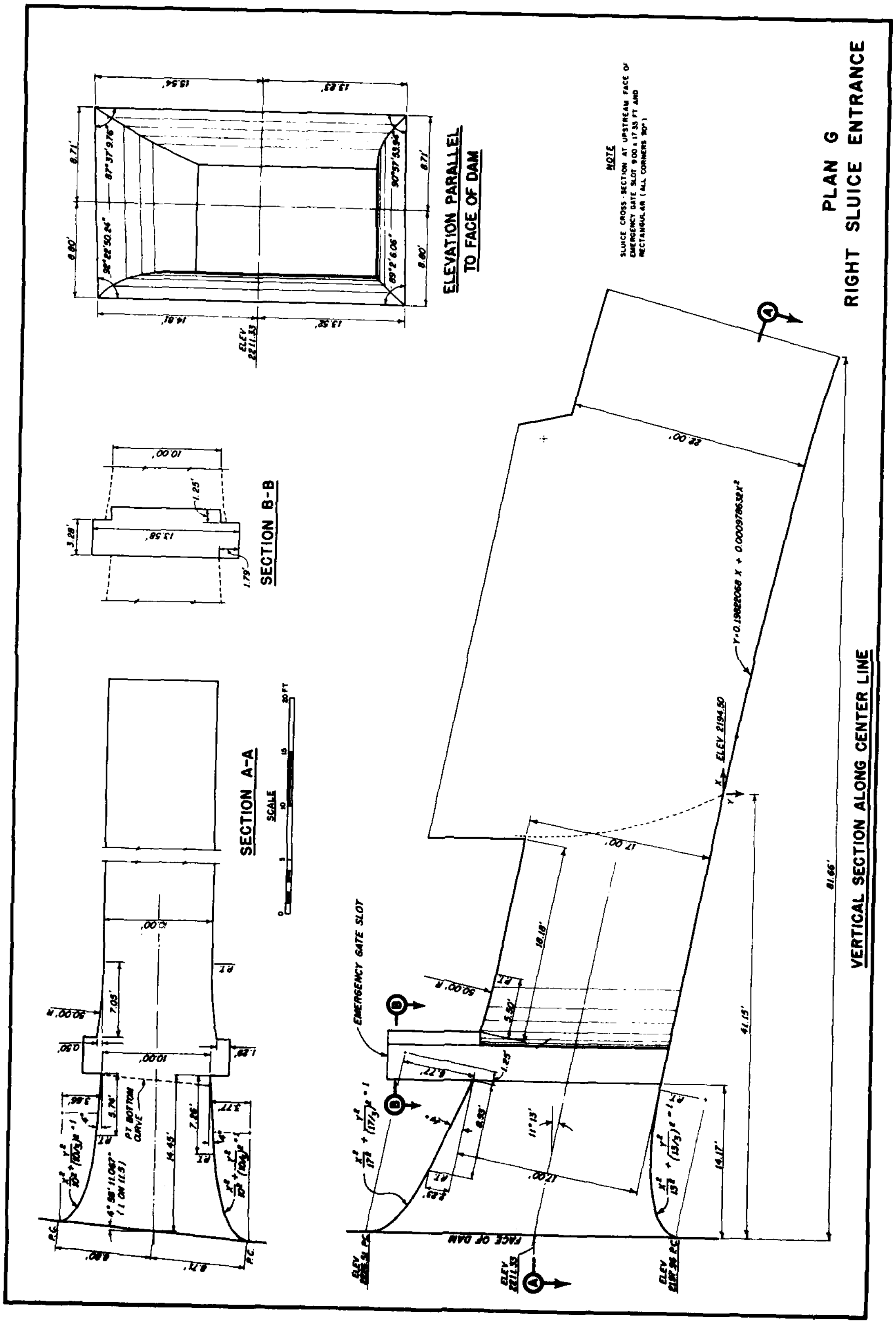
PIEZ NO.	STATION	ELEV	PIEZ NO	STATION	ELEV
32F	1+15.8	2199.7	46F	1+19.0	2207.6
33F	1+17.5	2201.2	47F	1+20.4	2207.4
34F	1+17.5	2207.9	48F	1+22.9	2206.8
35F	1+15.8	2199.7	49F	1+23.9	2206.6
36F	1+17.5	2201.2	50F	1+27.3	2206.0
37F	1+17.5	2207.9	51F	1+31.3	2205.2
38F	1+18.0	2199.3	52F	1+18.0	2214.5
39F	1+19.0	2199.1	53F	1+19.0	2214.3
40F	1+20.4	2198.8	54F	1+20.4	2214.0
41F	1+22.9	2198.3	55F	1+22.9	2213.5
42F	1+23.9	2198.1	56F	1+23.9	2213.3
43F	1+27.3	2197.5	57F	1+27.3	2212.7
44F	1+31.3	2196.7	58F	1+31.3	2211.9
45F	1+18.0	2207.8			

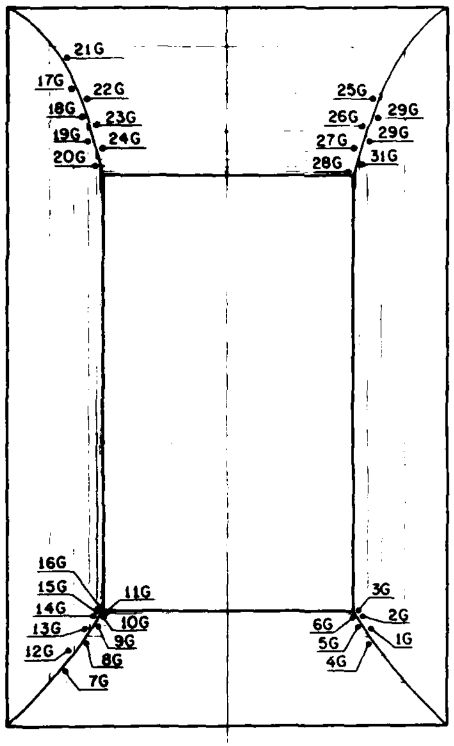
NOTE
ENTRANCE DETAILS SHOWN ON PLATE 37



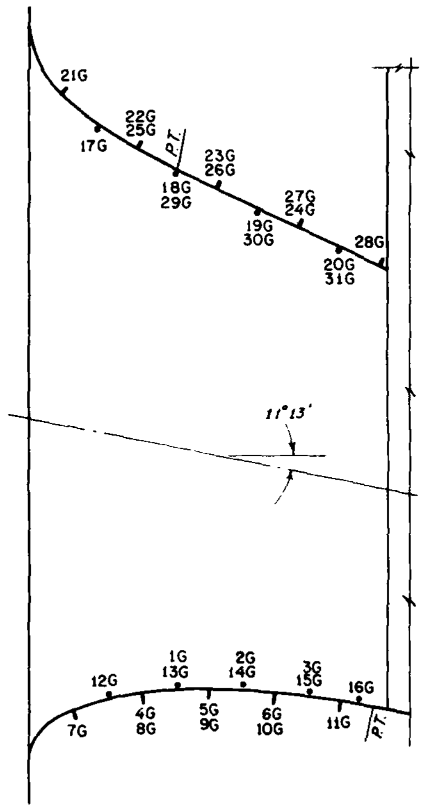
ELEVATION

PIEZOMETER LOCATIONS
PLAN F BELLMOUTH TO VALVE


PLAN G
RIGHT SLUICE ENTRANCE
ELEVATION PARALLEL TO FACE OF DAM
SECTION B-B
SECTION A-A
SCALE
VERTICAL SECTION ALONG CENTER LINE
EMERGENCY GATE SLOT
FACE OF DAM
ELEV 2194.50
ELEV 2211.33
22.00'
17.00'
81.66'
41.15'
14.17'
18.18'
10.00'
13.58'



FACE VIEW NORMAL TO CENTER LINE

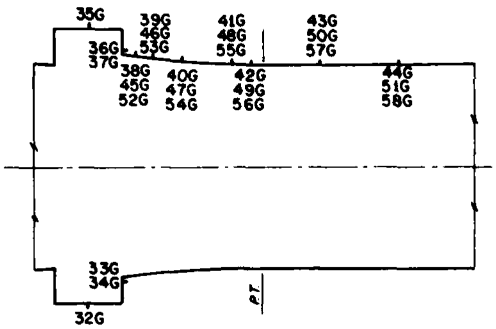


SECTION ALONG CENTER LINE

PIEZOMETER LOCATIONS					
PIEZ NO.	STATION	ELEV	PIEZ NO.	STATION	ELEV
1G	1+05.3	2200.9	17G	1+03.2	2222.4
2G	1+08.0	2200.9	18G	1+06.4	2220.7
3G	1+10.6	2200.6	19G	1+09.5	2219.2
			20G	1+12.7	2217.7
4G	1+04.0	2200.6			
5G	1+06.7	2200.7	21G	1+01.8	2223.8
6G	1+09.3	2200.6	22G	1+04.8	2221.7
			23G	1+07.9	2220.1
7G	1+02.3	2199.7	24G	1+11.1	2218.6
8G	1+05.0	2200.4			
9G	1+07.6	2200.5	25G	1+03.8	2222.2
10G	1+10.2	2200.4	26G	1+07.0	2220.6
11G	1+12.8	2200.1	27G	1+10.2	2219.1
			28G	1+13.5	2217.5
12G	1+03.6	2200.3			
13G	1+06.3	2200.7	29G	1+05.4	2221.2
14G	1+08.9	2200.7	30G	1+08.6	2219.6
15G	1+11.5	2200.4	31G	1+11.8	2218.1
16G	1+13.4	2200.2			

NOTE
ENTRANCE DETAILS SHOWN ON PLATE 40.

PIEZOMETER LOCATIONS
PLAN G RIGHT SLUICE ENTRANCE

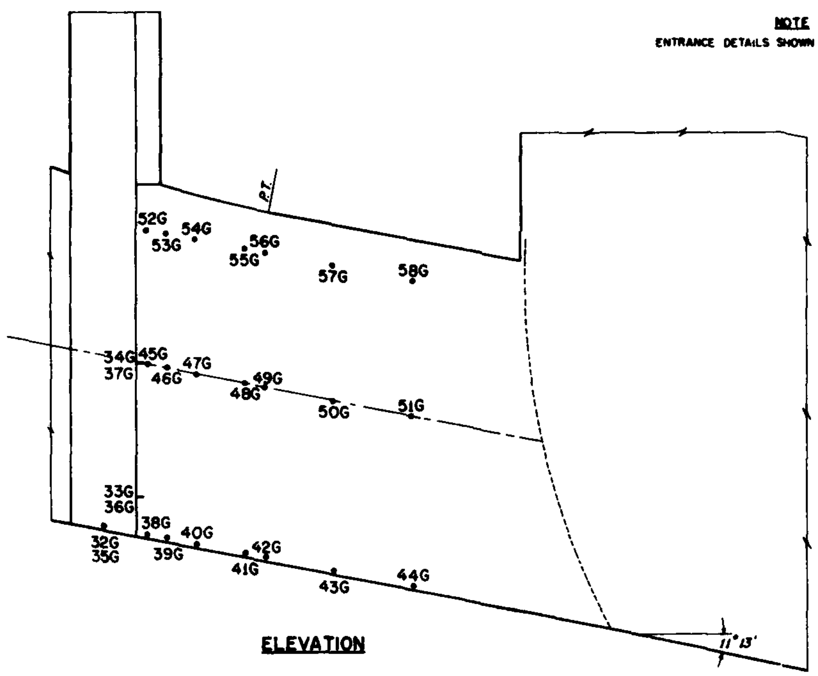


PLAN

PIEZOMETER LOCATIONS					
PIEZ NO.	STATION	ELEV	PIEZ NO.	STATION	ELEV
32G	1+15.8	2199.7	46G	1+19.0	2207.6
33G	1+17.5	2201.2	47G	1+20.4	2207.4
34G	1+17.5	2207.9	48G	1+22.9	2206.8
35G	[illegible] 5.8	2199.7	49G	1+23.9	2206.6
36G	1+17.5	2201.2	50G	1+27.3	2206.0
37G	1+17.5	2207.9	51G	1+31.3	2205.2
38G	1+18.0	2199.3	52G	1+18.0	2214.5
39G	1+19.0	2199.1	53G	1+19.0	2214.3
40G	1+20.4	2198.8	54G	1+20.4	2214.0
41G	1+22.9	2198.3	55G	1+22.9	2213.5
42G	1+23.9	2198.1	56G	1+23.9	2213.3
43G	1+27.3	2197.5	57G	1+27.3	2212.7
44G	1+31.3	2196.7	58G	1+31.3	2211.9
45G	1+18.0	2207.8			

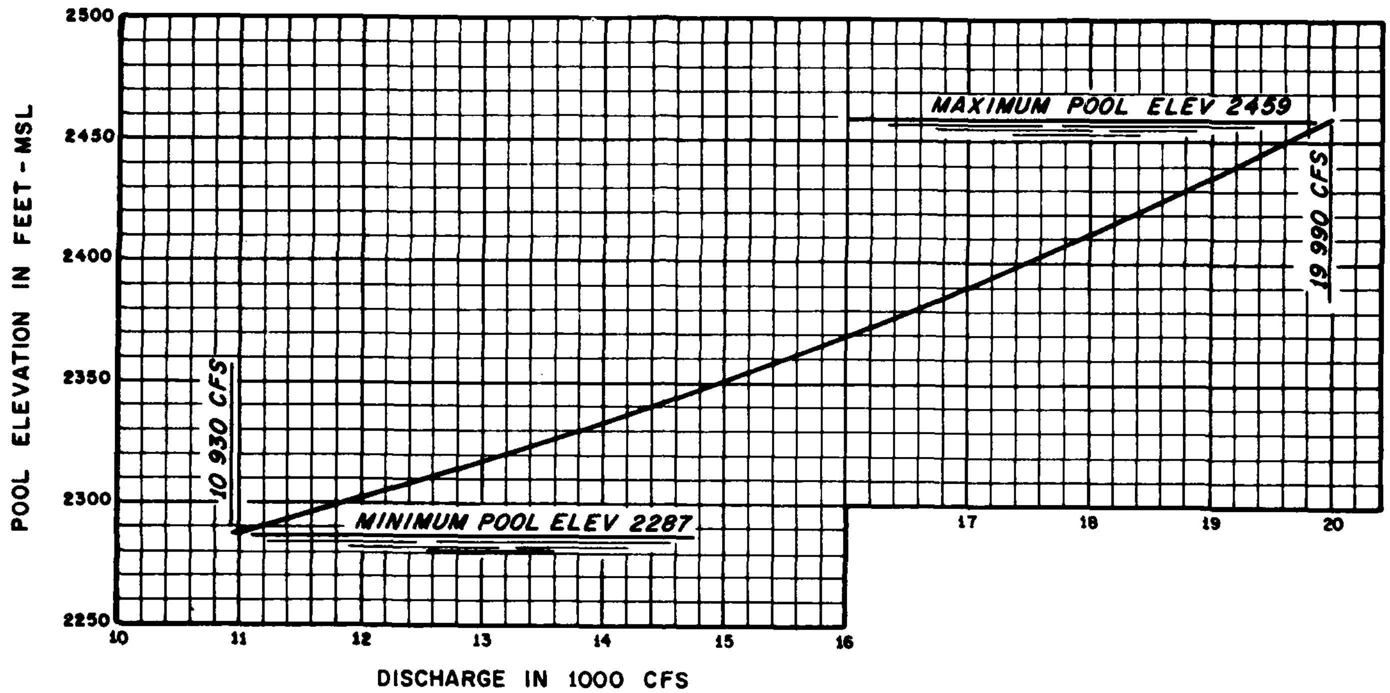
NOTE

ENTRANCE DETAILS SHOWN ON PLATE 40.



ELEVATION

PIEZOMETER LOCATIONS
PLAN G BELLMOUTH TO VALVE

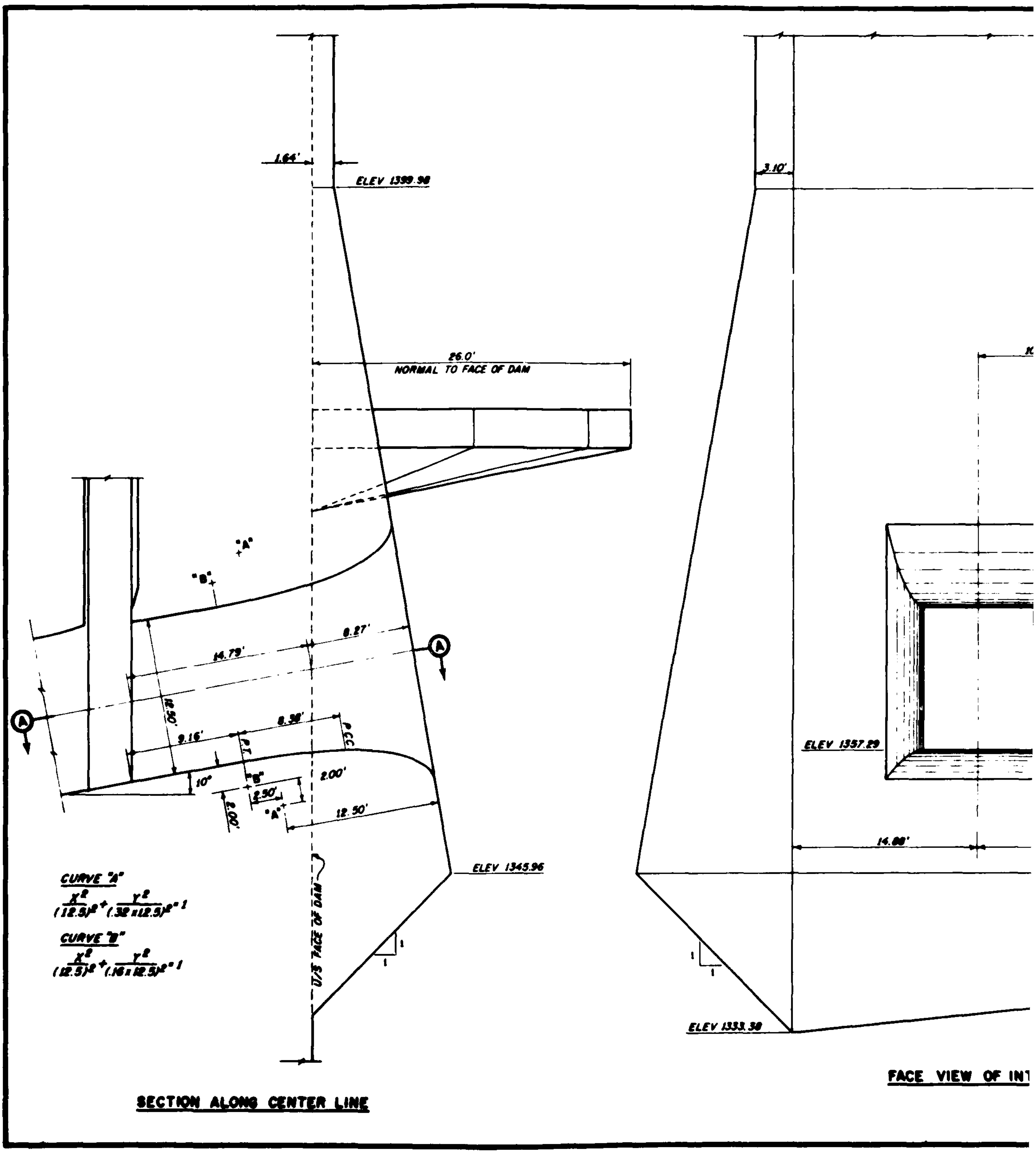


NOTE

DETAILS OF BELLMOUTH AND VALVE SECTION SHOWN ON PLATE 40.

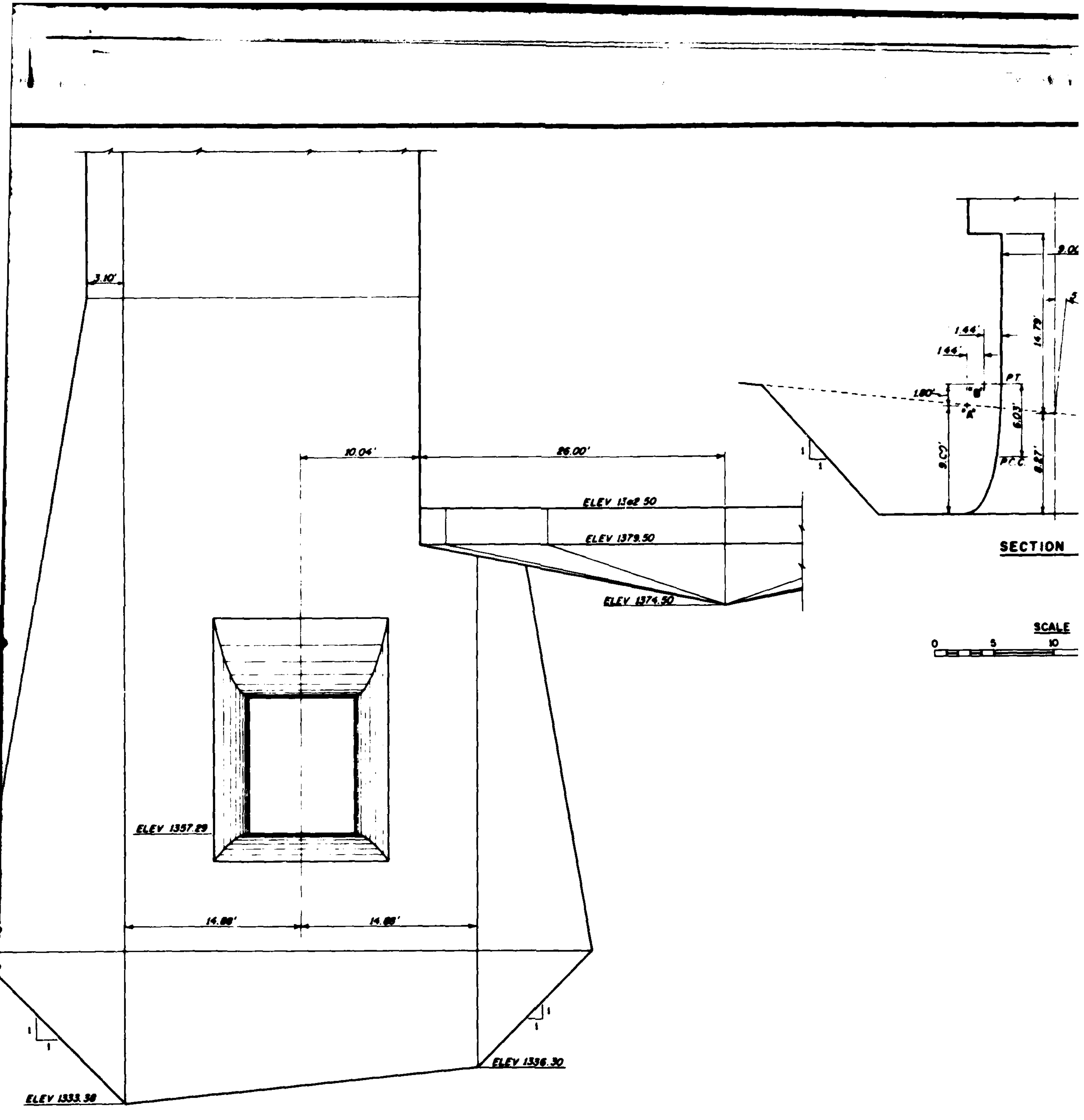
DISCHARGE RATING

FULLY OPEN VALVE
PLAN G RIGHT SLUICE

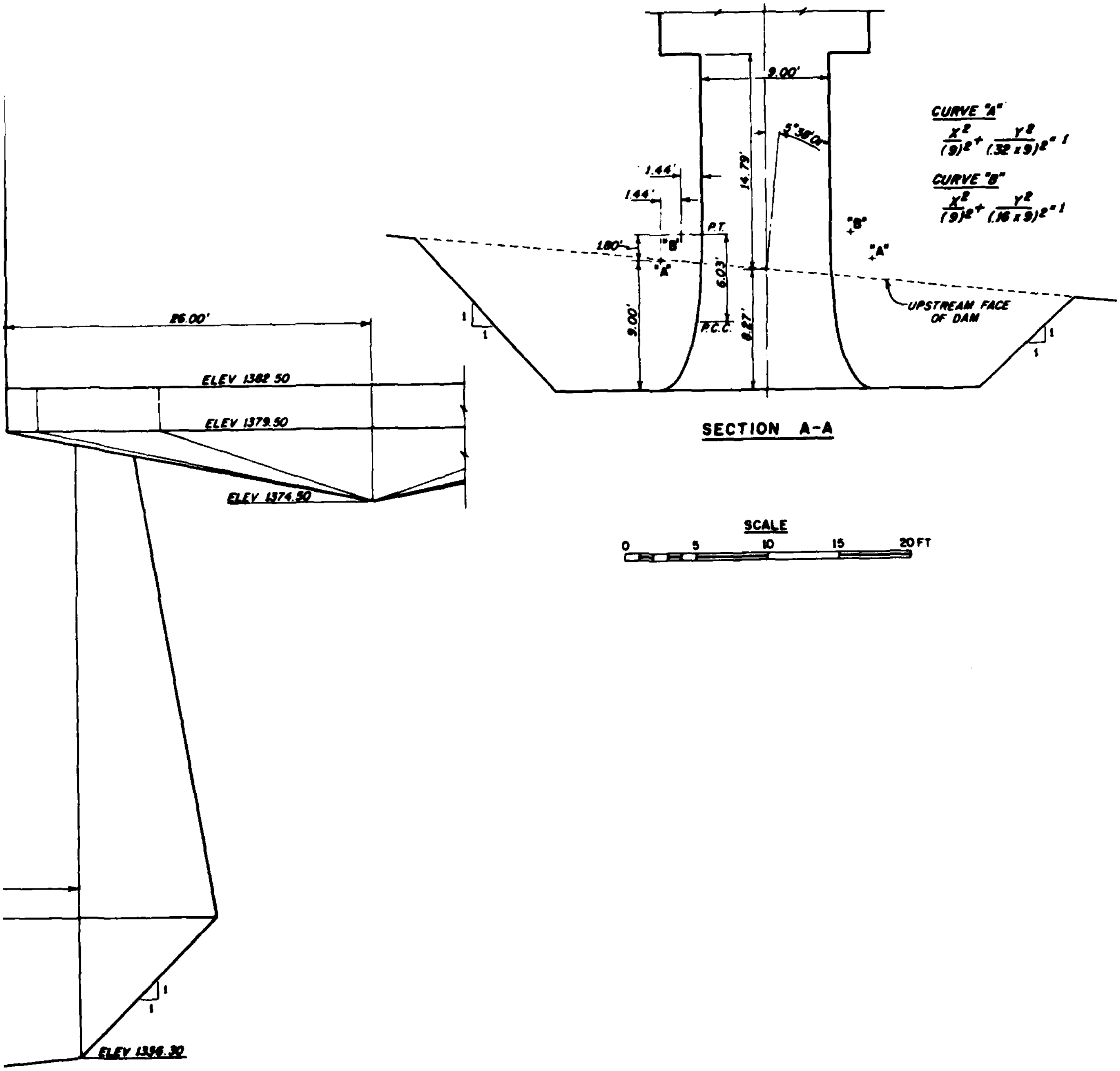


SECTION ALONG CENTER LINE

FACE VIEW OF INT

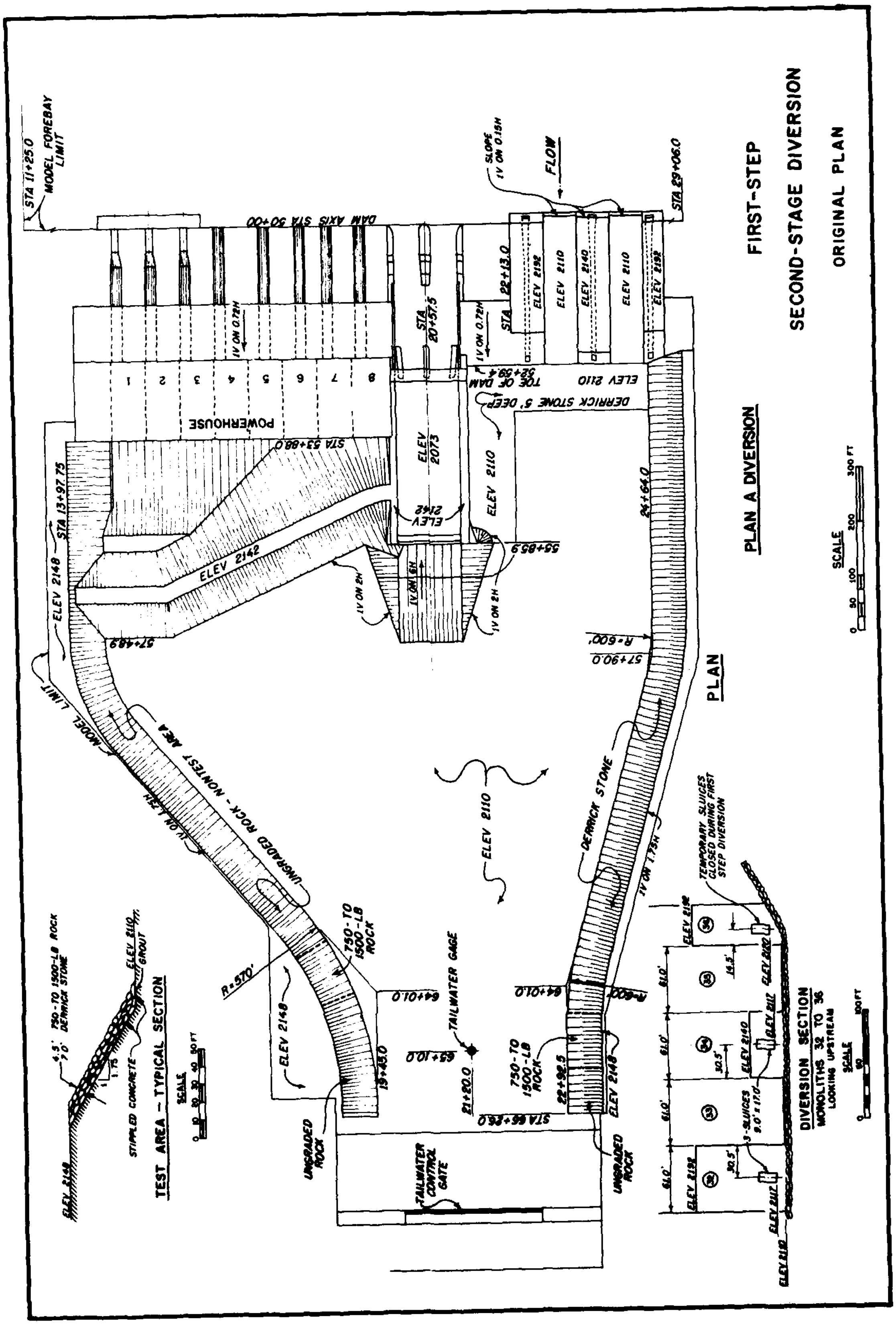


FACE VIEW OF INTAKE

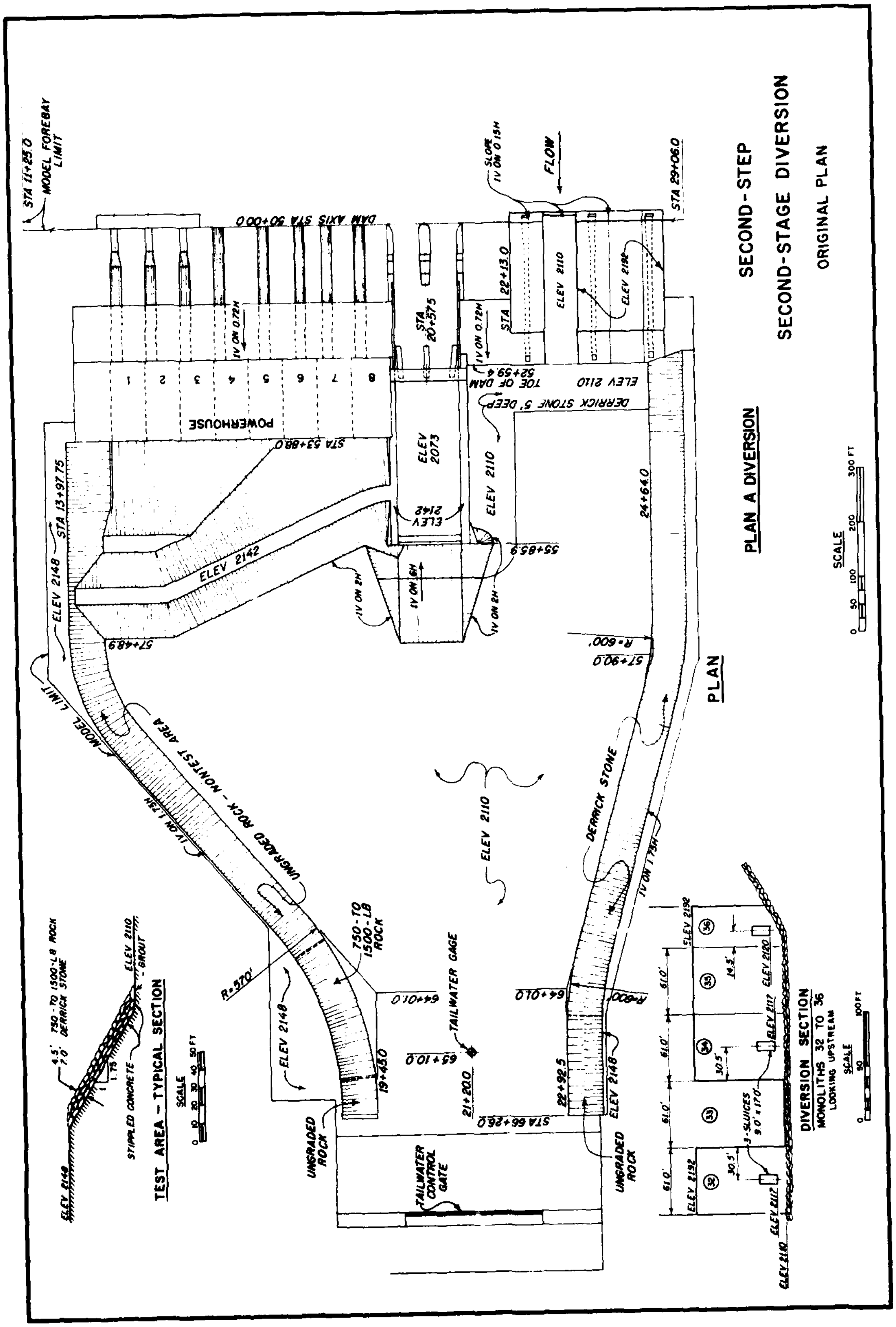


DWORSHAK DAM

PLAN D RIGHT CONDUIT ENTRANCE



FIRST-STEP
SECOND-STAGE DIVERSION
ORIGINAL PLAN
PLAN A DIVERSION
PLAN
SCALE
STA 11+25.0
MODEL FOREBAY LIMIT
DAM AXIS STA 50+00
SLOPE 1V ON 0.15H
FLOW
STA 29+06.0
STA 22+13.0
ELEV 2192
ELEV 2110
ELEV 2140
ELEV 2110
ELEV 2198
STA 20+57.5
1V ON 0.72H
POWERHOUSE
TOE OF DAM 52+59.4
ELEV 2110
DERRICK STONE 5' DEEP
STA 53+88.0
ELEV 2073
ELEV 2110
ELEV 2142
55+85.9
24+64.0
STA 13+97.75
ELEV 2148
ELEV 2142
1V ON 2H
1V ON 2H
57+48.9
R=600'
57+90.0
MODEL LIMIT
UNGRADED ROCK - NONTEST AREA
1V ON 1.75H
ELEV 2110
DERRICK STONE
1V ON 1.75H
TEMPORARY SLUICES CLOSED DURING FIRST STEP DIVERSION
750-TO 1500-LB ROCK
R=570'
TAILWATER GAGE
64+01.0
64+01.0
R=600'
ELEV 2148
65+10.0
750-TO 1500-LB ROCK
19+45.0
21+80.0
22+92.5
ELEV 2148
STA 66+26.0
UNGRADED ROCK
TAILWATER CONTROL GATE
UNGRADED ROCK
TEST AREA - TYPICAL SECTION
750-TO 1500-LB ROCK DERRICK STONE
STIPPLED CONCRETE
GROUT
DIVERSION SECTION
MONOLITHS 32 TO 36
LOOKING UPSTREAM
3-SLUICES 9.0' x 17.0'


SECOND-STEP
SECOND-STAGE DIVERSION
ORIGINAL PLAN
PLAN A DIVERSION
PLAN
SCALE
0 50 100 200 300 FT
STA 11+25.0 MODEL FOREBAY LIMIT
DAM AXIS STA 50+00.0
STA 29+06.0
FLOW
SLOPE 1V ON 0.15H
STA 22+13.0
ELEV 2110
ELEV 2192
STA 20+57.5
1V ON 0.72H
POWERHOUSE
1 2 3 4 5 6 7 8
52+59.4 TOE OF DAM
ELEV 2110
DERRICK STONE 5' DEEP
STA 53+88.0
ELEV 2073
ELEV 2110
ELEV 2142
ELEV 2142
STA 13+97.75
ELEV 2148
55+85.9
1V ON 2H
1V ON 1.5H
1V ON 2H
24+64.0
57+48.9
R=600'
57+90.0
MODEL LIMIT
UNGRADED ROCK - NONTEST AREA
1V ON 1.75H
DERRICK STONE
ELEV 2110
1V ON 1.75H
750- TO 1500-LB ROCK
R=570'
TAILWATER GAGE
64+01.0
64+01.0
R=600'
ELEV 2148
65+10.0
19+43.0
21+20.0
22+92.5
ELEV 2148
STA 66+26.0
UNGRADED ROCK
UNGRADED ROCK
TAILWATER CONTROL GATE
TEST AREA - TYPICAL SECTION
4.5' 7.0'
750- TO 1500-LB ROCK DERRICK STONE
ELEV 2110
GROUT
STIPPLED CONCRETE
ELEV 2148
SCALE
0 10 20 30 40 50 FT
DIVERSION SECTION
MONOLITHS 32 TO 36
LOOKING UPSTREAM
SCALE
0 50 100 FT
ELEV 2192
36
35
34
33
32
61.0'
14.5'
30.5'
ELEV 2120
ELEV 2117
3-SLUICES 9.0' x 17.0'
ELEV 2192
ELEV 2117
ELEV 2110

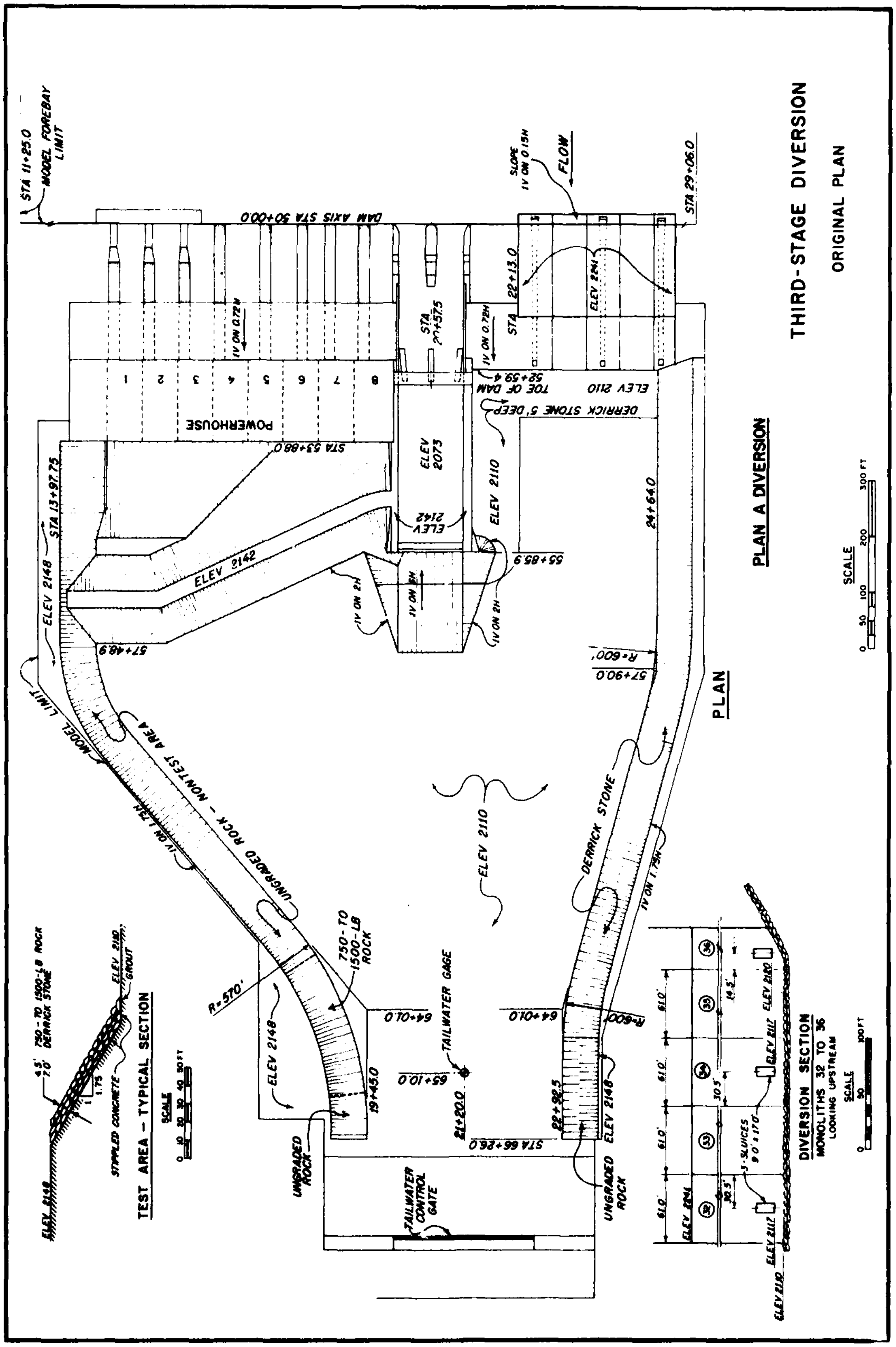
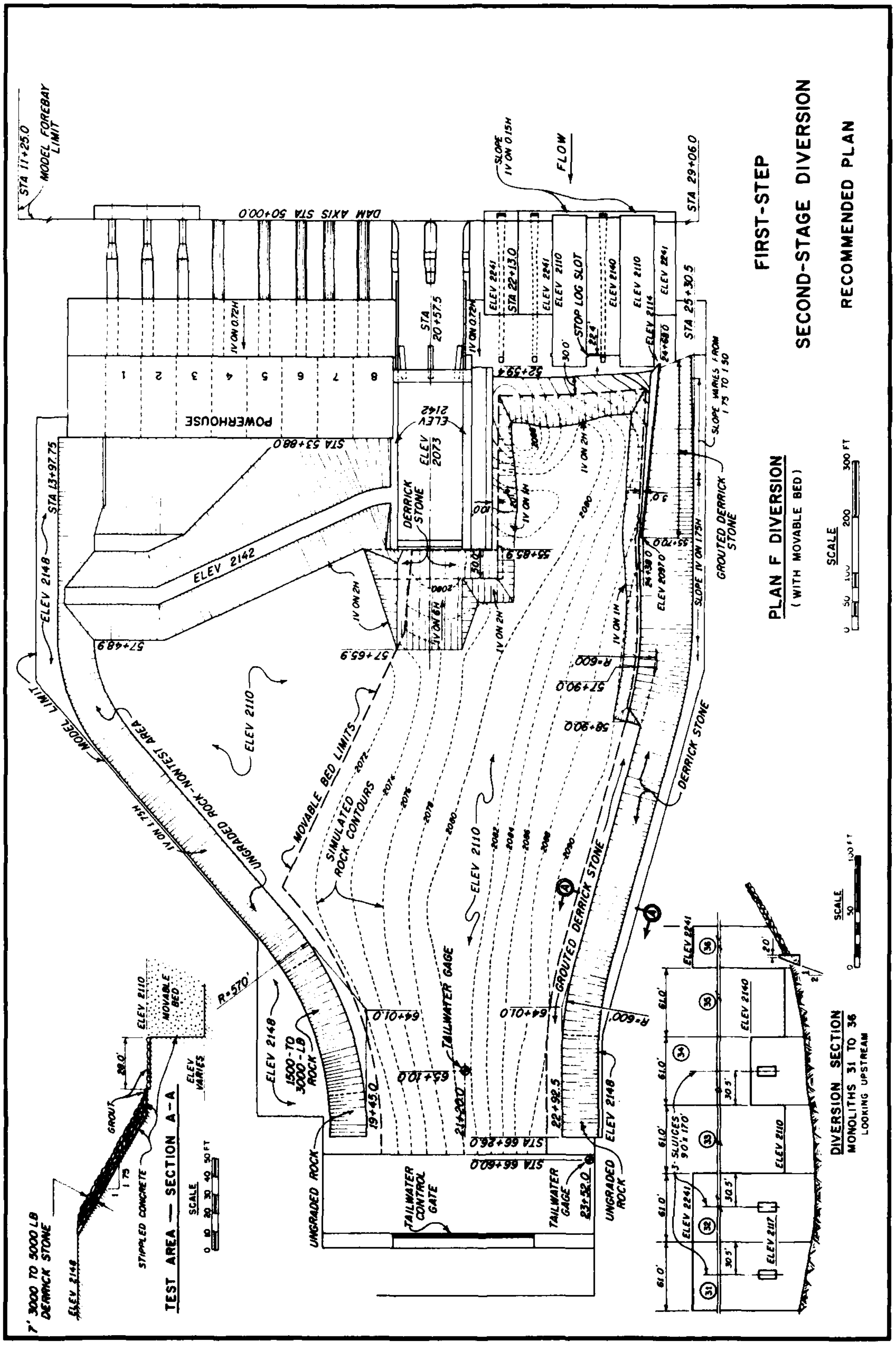
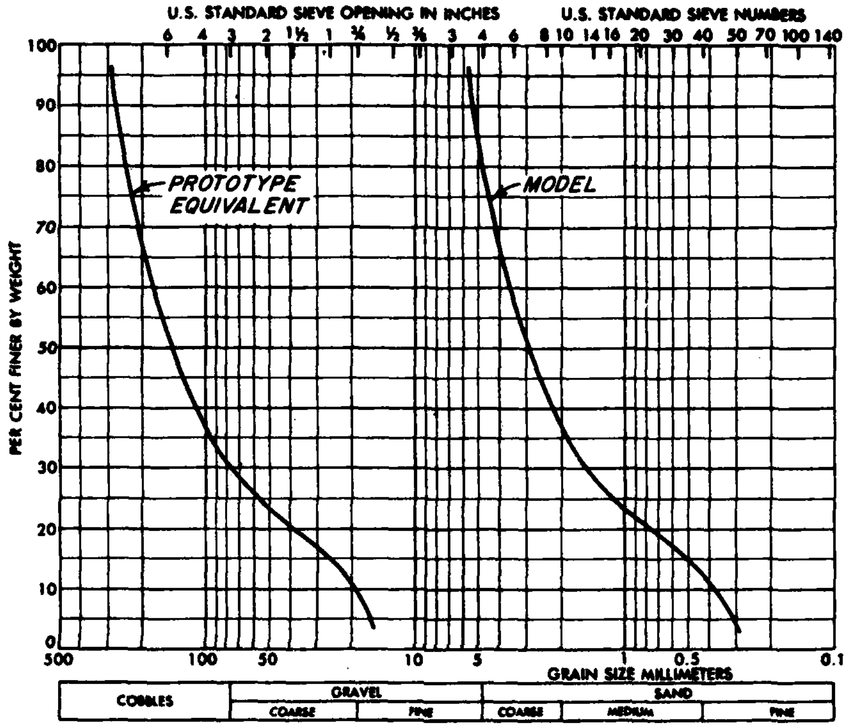
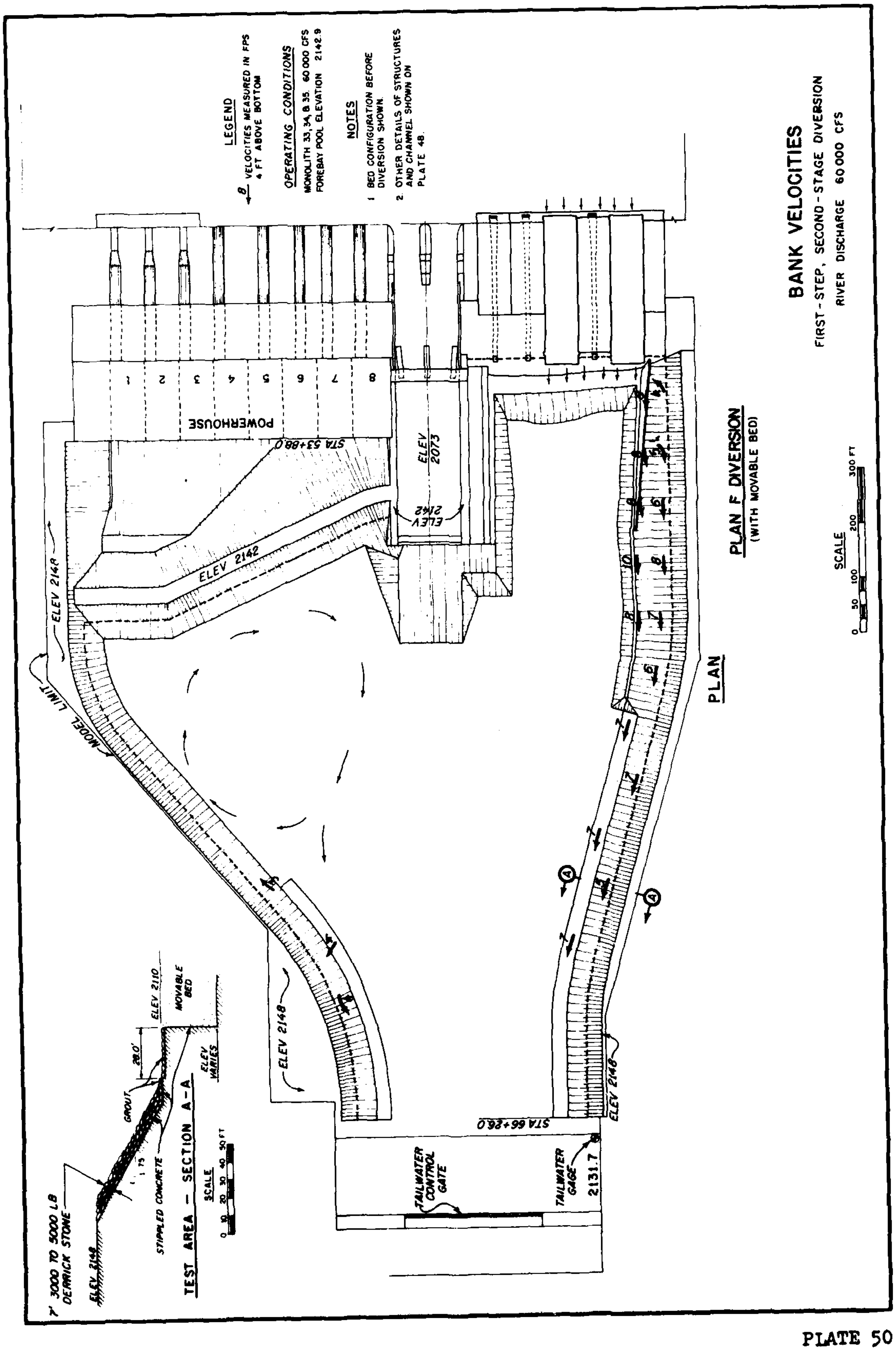

THIRD-STAGE DIVERSION
ORIGINAL PLAN
PLAN A DIVERSION
PLAN
STA 11+25.0
MODEL FOREBAY LIMIT
DAM AXIS STA 50+00.0
SLOPE 1V ON 0.15H
FLOW
STA 29+06.0
STA 22+13.0
ELEV 2241
1V ON 0.72H
POWERHOUSE
1
2
3
4
5
6
7
8
TOE OF DAM
52+59.4
ELEV 2110
DERRICK STONE 5' DEEP
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2148
STA 13+97.75
55+85.9
1V ON 2H
57+48.9
R=600'
57+90.0
24+64.0
MODEL LIMIT
UNGRADED ROCK - NONTEST AREA
1V ON 1.75H
DERRICK STONE
750-TO 1500-LB ROCK
R=570'
64+00.0
TAILWATER GAGE
65+10.0
19+45.0
21+20.0
22+92.5
STA 66+26.0
UNGRADED ROCK
TAILWATER CONTROL GATE
SCALE
0
50
100
200
300 FT
TEST AREA - TYPICAL SECTION
750-TO 1500-LB ROCK DERRICK STONE
4.5'
7.0'
ELEV 2110
GROUT
STIPPLED CONCRETE
ELEV 2148
DIVERSION SECTION
MONOLITHS 32 TO 36
LOOKING UPSTREAM
61.0'
30.5'
14.5'
3-SLUICES 9.0' x 17.0'
ELEV 2241
ELEV 2112
ELEV 2120
ELEV 2110
32
33
34
35
36

PLATE 47

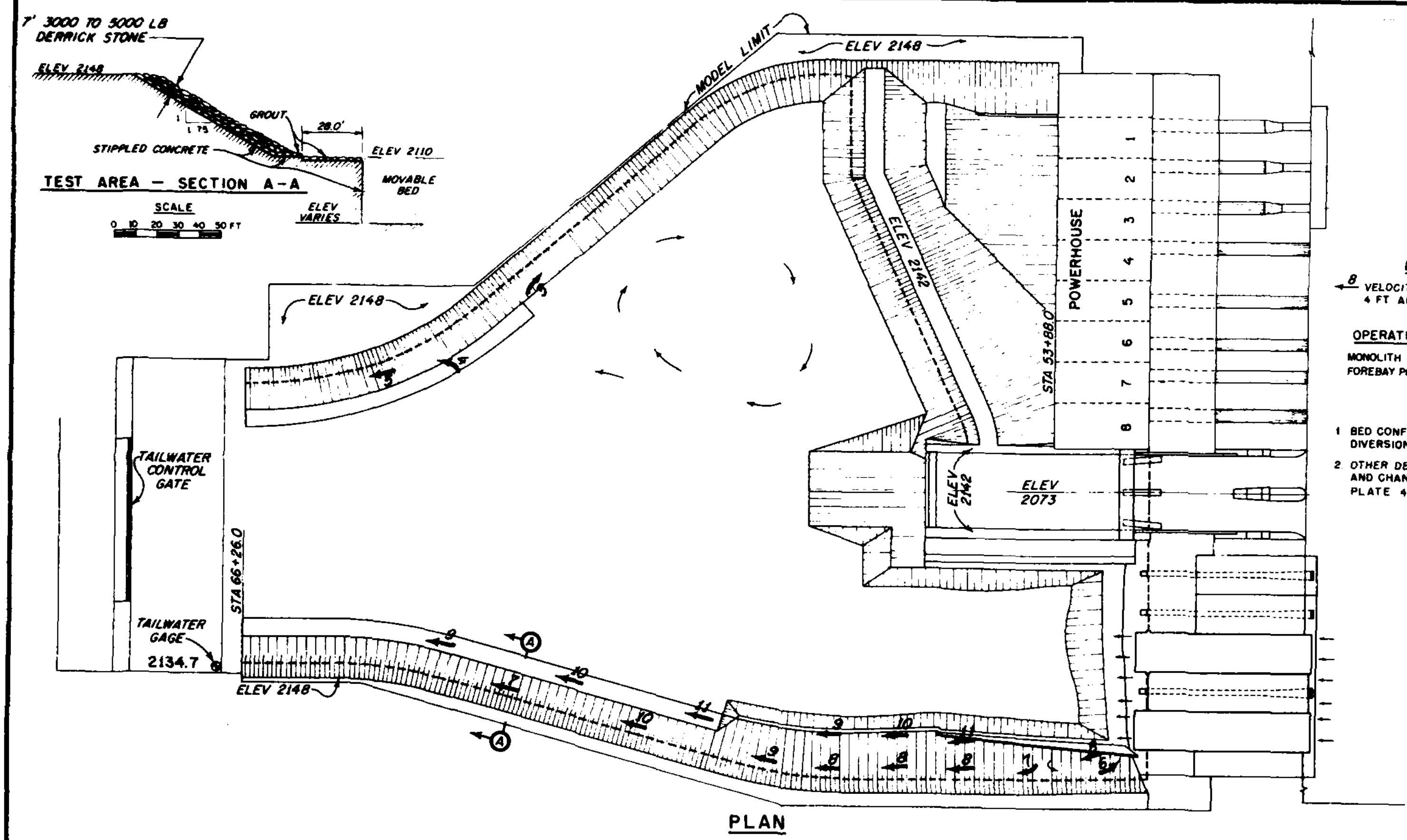

FIRST-STEP
SECOND-STAGE DIVERSION
RECOMMENDED PLAN
PLAN F DIVERSION
(WITH MOVABLE BED)
SCALE
FLOW
DAM AXIS STA 50+00.0
POWERHOUSE
STA 11+25.0
MODEL FOREBAY LIMIT
STA 29+06.0
STA 25+30.5
STA 22+13.0
STOP LOG SLOT
STA 20+57.5
DERRICK STONE
ELEV 2073
ELEV 2142
ELEV 2148
ELEV 2110
MODEL LIMIT
UNGRADED ROCK-NONTEST AREA
MOVABLE BED LIMITS
SIMULATED ROCK CONTOURS
GROUTED DERRICK STONE
DERRICK STONE
TAILWATER GAGE
TAILWATER CONTROL GATE
UNGRADED ROCK
1500-TO 3000-LB ROCK
R=570'
R=600'
TEST AREA — SECTION A-A
7' 3000 TO 5000 LB DERRICK STONE
STIPPLED CONCRETE
MOVABLE BED
ELEV VARIES
DIVERSION SECTION
MONOLITHS 31 TO 36
LOOKING UPSTREAM
3-SLUICES 9.0'x 17.0'



GRAIN-SIZE DISTRIBUTION
MOVABLE BED TEST



BANK VELOCITIES
FIRST-STEP, SECOND-STAGE DIVERSION
RIVER DISCHARGE 60 000 CFS
LEGEND
8 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
MONOLITH 33, 34, & 35, 60 000 CFS
FOREBAY POOL ELEVATION 2142.9
NOTES
1 BED CONFIGURATION BEFORE DIVERSION SHOWN.
2 OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48.
POWERHOUSE
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2148
MODEL LIMIT
STA 66+26.0
TAILWATER CONTROL GATE
TAILWATER GAGE
2131.7
PLAN F DIVERSION
(WITH MOVABLE BED)
PLAN
SCALE
0 50 100 200 300 FT
TEST AREA — SECTION A-A
3000 TO 5000 LB DERRICK STONE
STIPPLED CONCRETE
GROUT
28.0'
ELEV 2110
MOVABLE BED
ELEV VARIES
0 10 20 30 40 50 FT



LEGEND

8 ← VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM

OPERATING CONDITIONS

MONOLITH 33, 34, & 35 82 000 CFS
FOREBAY POOL ELEVATION 2149.2

NOTES

1 BED CONFIGURATION BEFORE DIVERSION SHOWN.

2 OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48.

PLAN F DIVERSION

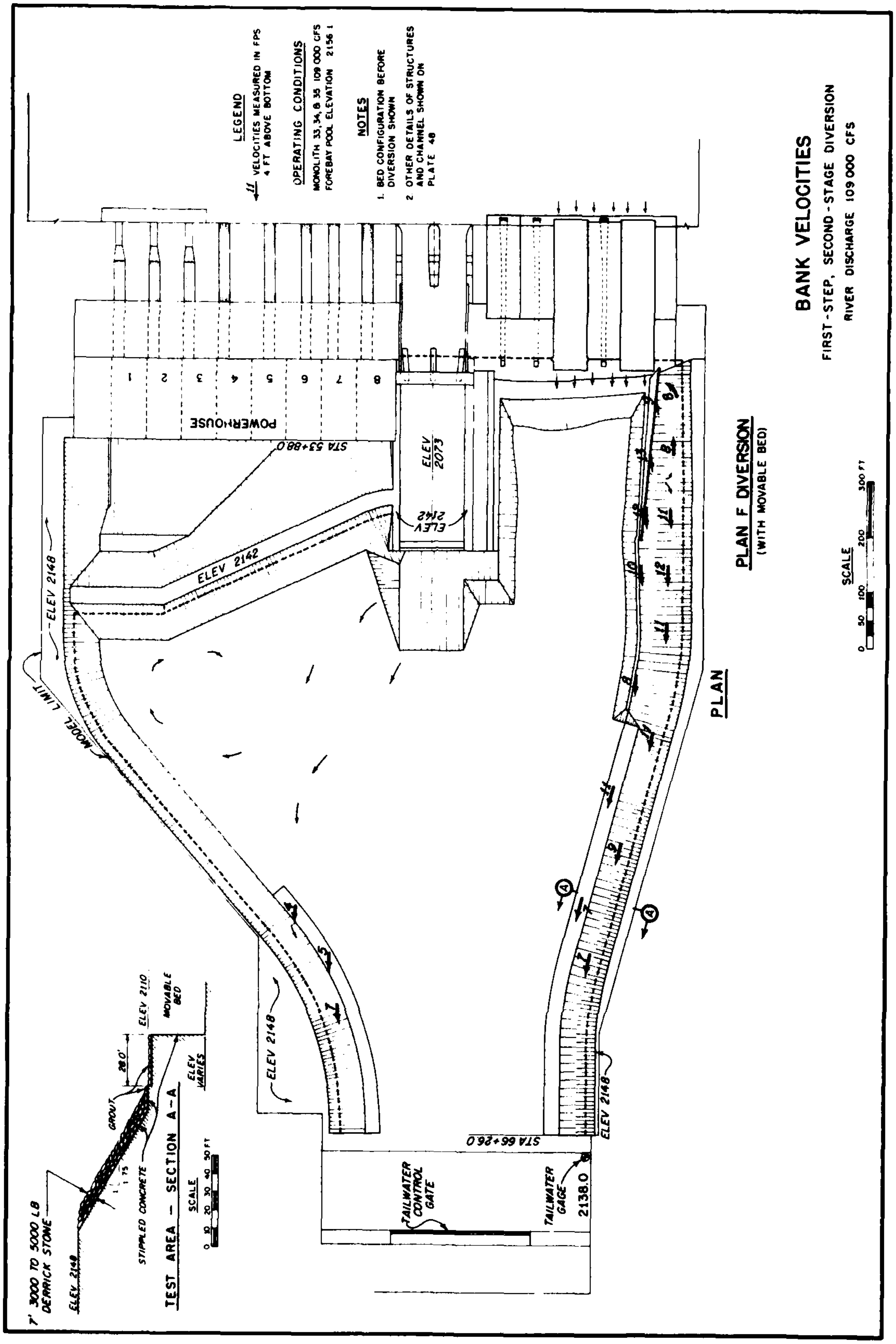
(WITH MOVABLE BED)

SCALE
0 50 100 200 300 FT

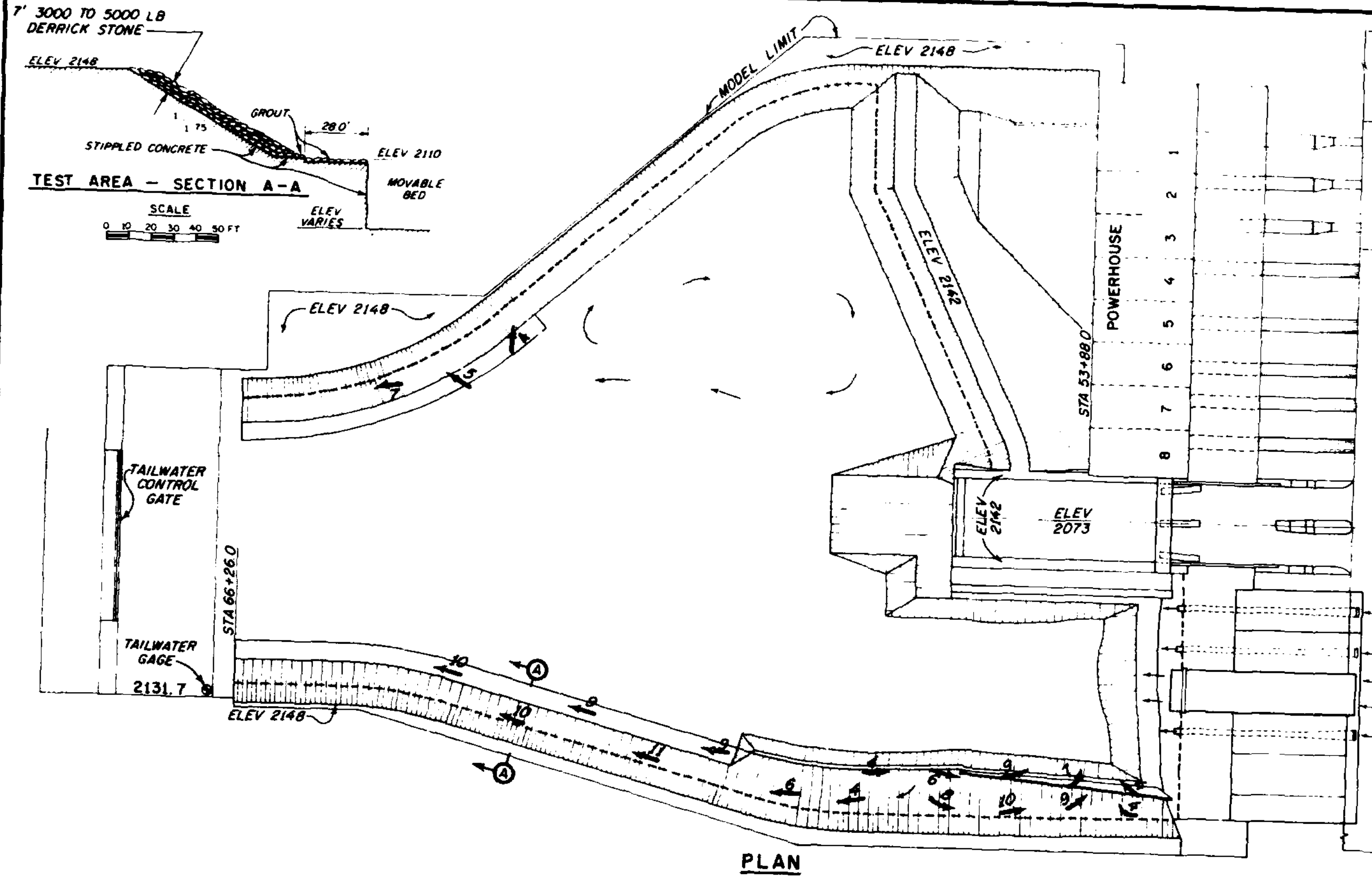
BANK VELOCITIES

FIRST-STEP, SECOND-STAGE DIVERSION

RIVER DISCHARGE 82 000 CFS



BANK VELOCITIES
FIRST-STEP, SECOND-STAGE DIVERSION
RIVER DISCHARGE 109 000 CFS
LEGEND
11 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
MONOLITH 33, 34, & 35 109 000 CFS
FOREBAY POOL ELEVATION 2156 ±
NOTES
1. BED CONFIGURATION BEFORE DIVERSION SHOWN
2. OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48
POWERHOUSE
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2148
MODEL LIMIT
STA 66+26.0
TAILWATER CONTROL GATE
TAILWATER GAGE
2138.0
PLAN F DIVERSION
(WITH MOVABLE BED)
PLAN
SCALE
0 50 100 200 300 FT
7' 3000 TO 5000 LB DERRICK STONE
ELEV 2148
STIPPLED CONCRETE
GROUT
28.0'
ELEV 2110
MOVABLE BED
ELEV VARIES
TEST AREA - SECTION A-A
SCALE
0 10 20 30 40 50 FT



LEGEND

4 ← VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM

OPERATING CONDITIONS

MONOLITH 33	44 100 CFS
SLUICES	15 900 CFS

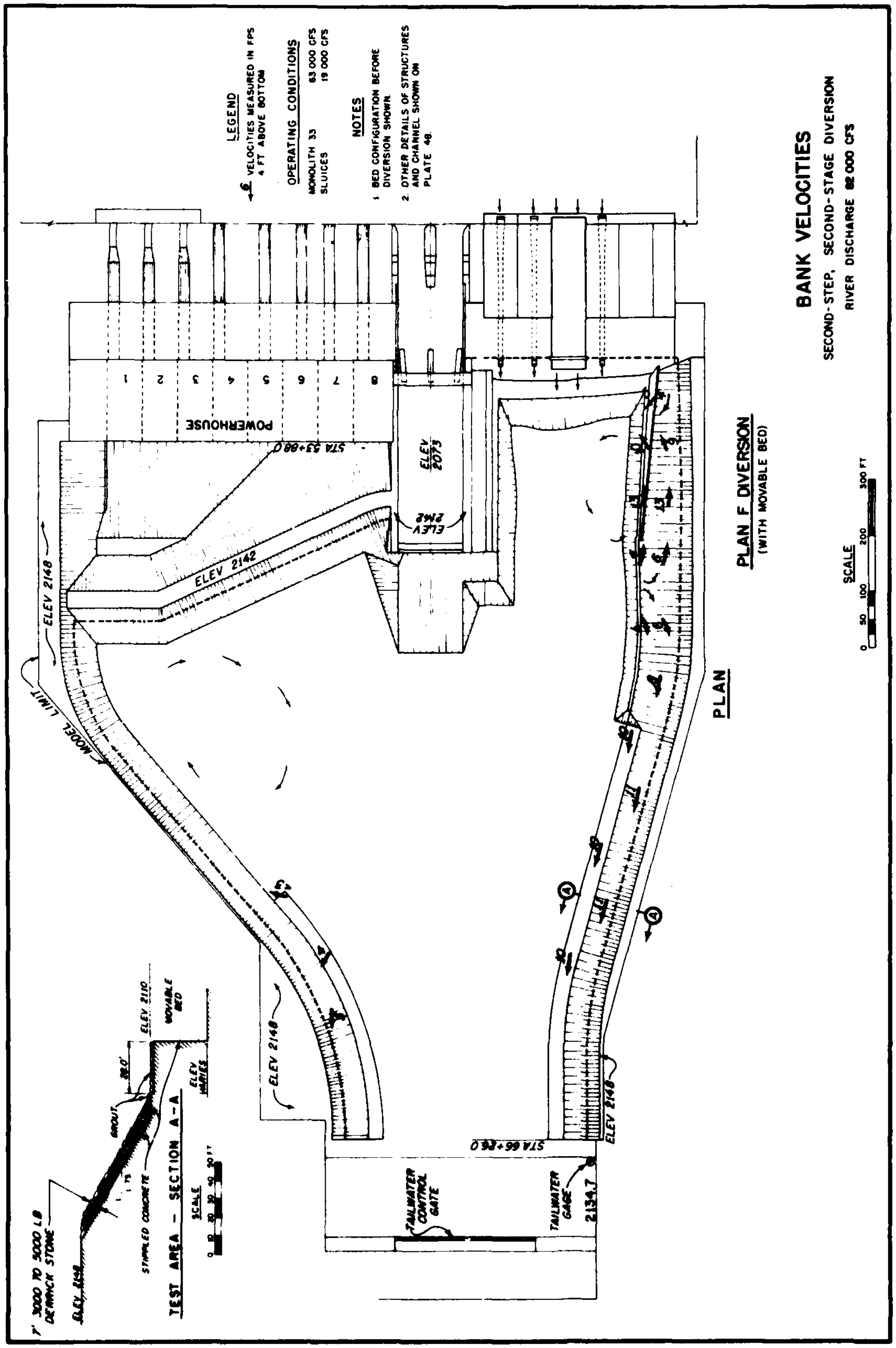
NOTES

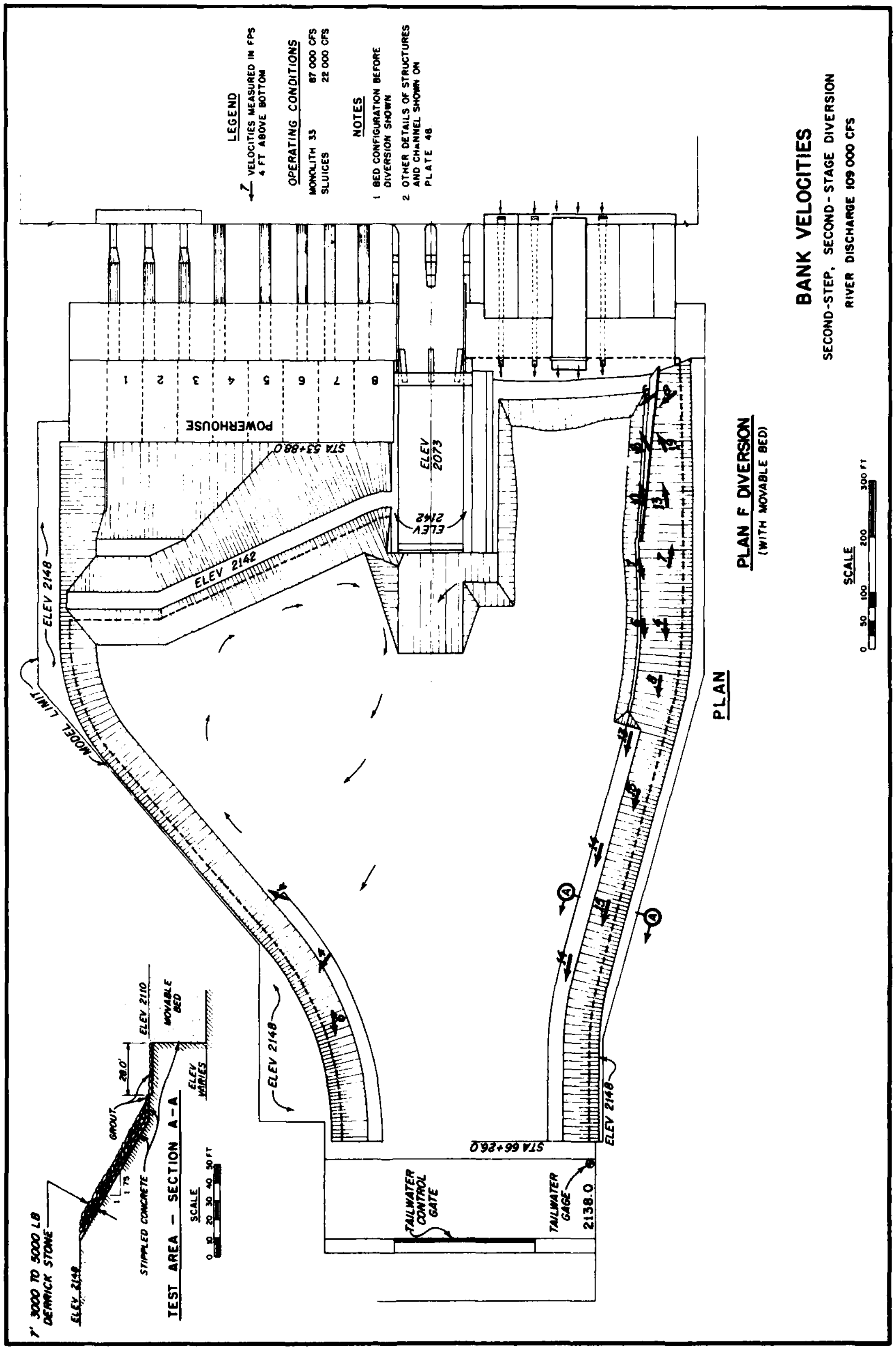
1. BED CONFIGURATION BEFORE DIVERSION SHOWN.
2. OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48

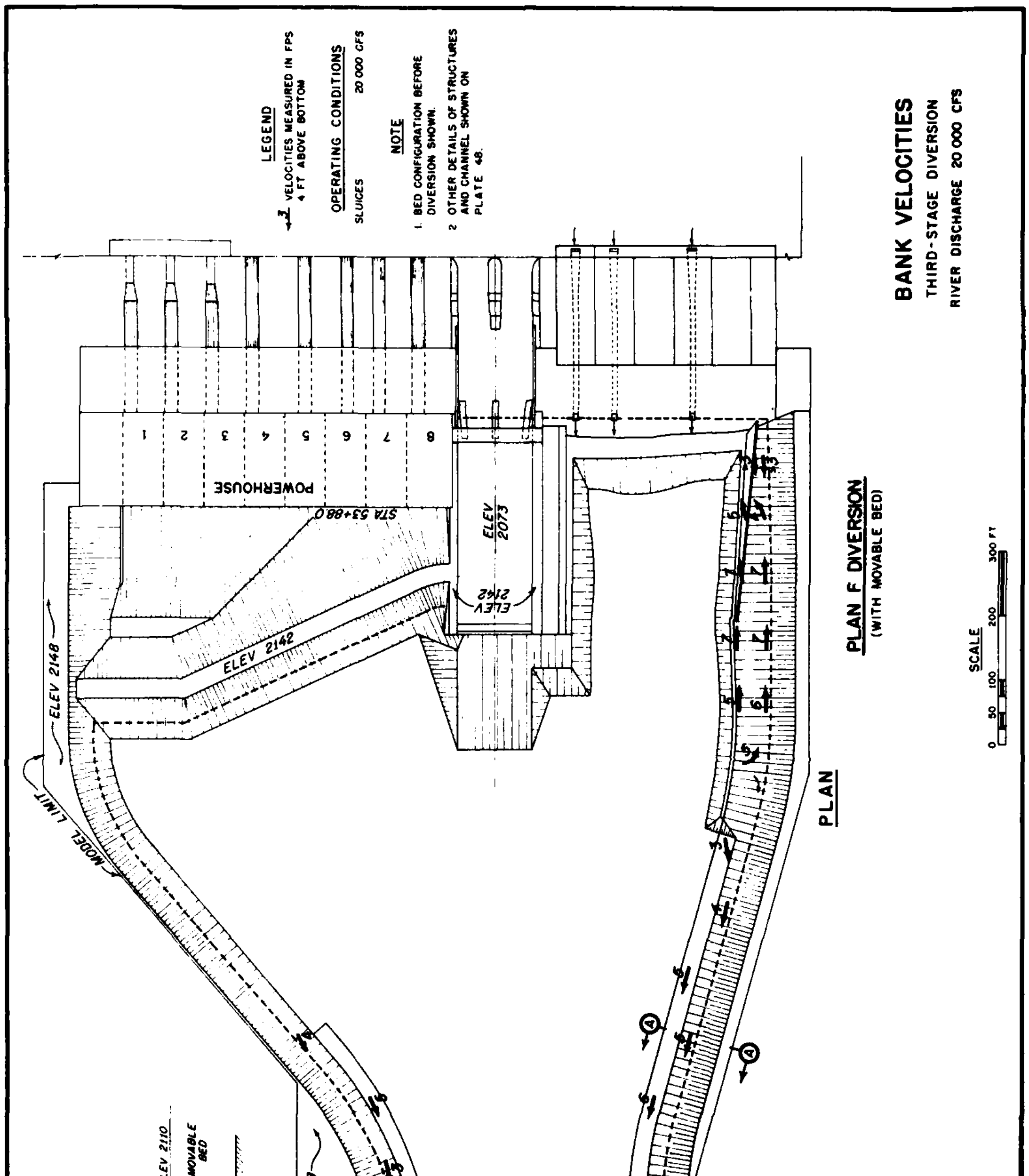
PLAN F DIVERSION
(WITH MOVABLE BED)

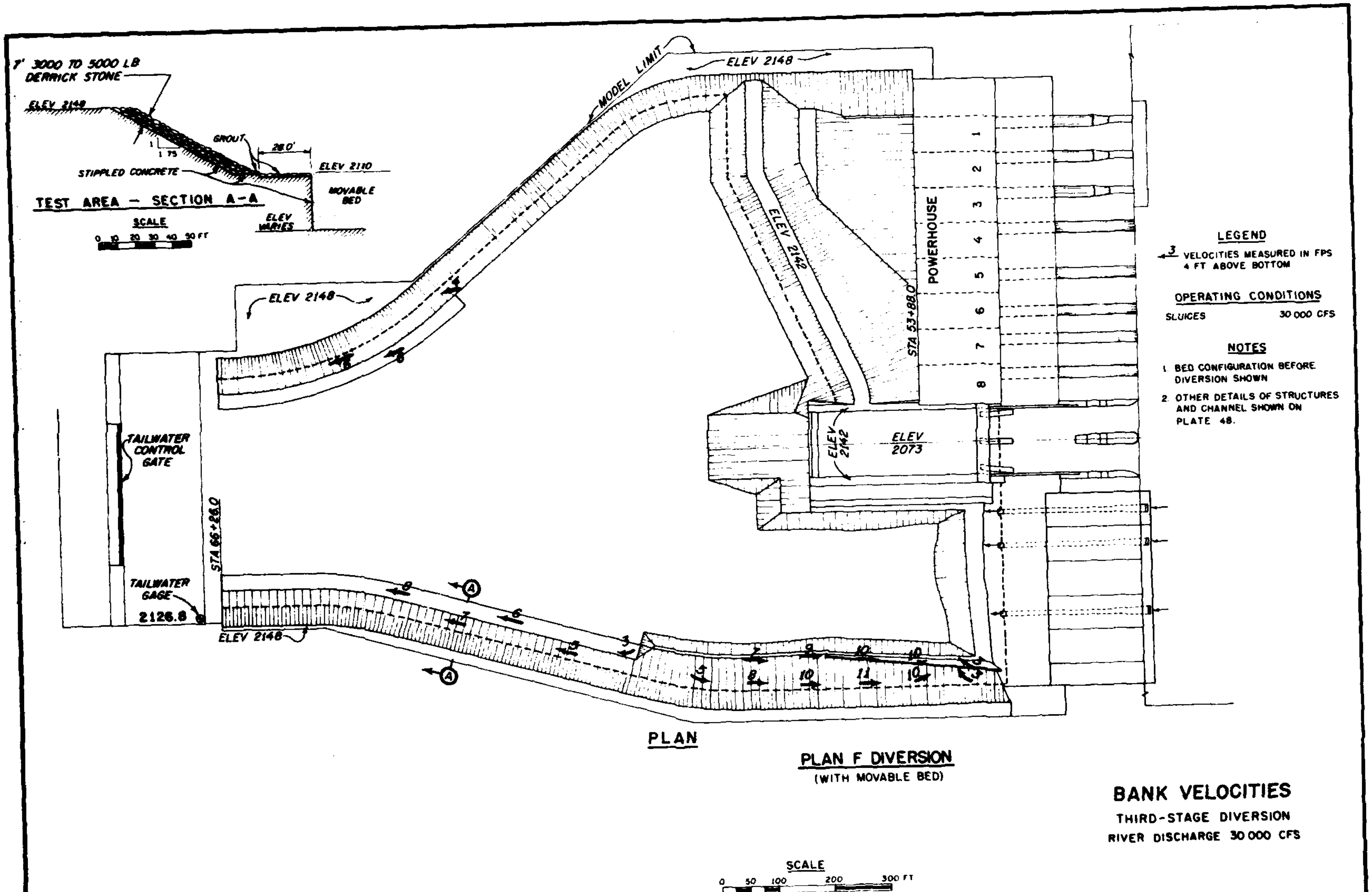
SCALE
0 50 100 200 300 FT

BANK VELOCITIES
SECOND-STEP, SECOND-STAGE DIVERSION
RIVER DISCHARGE 60 000 CFS

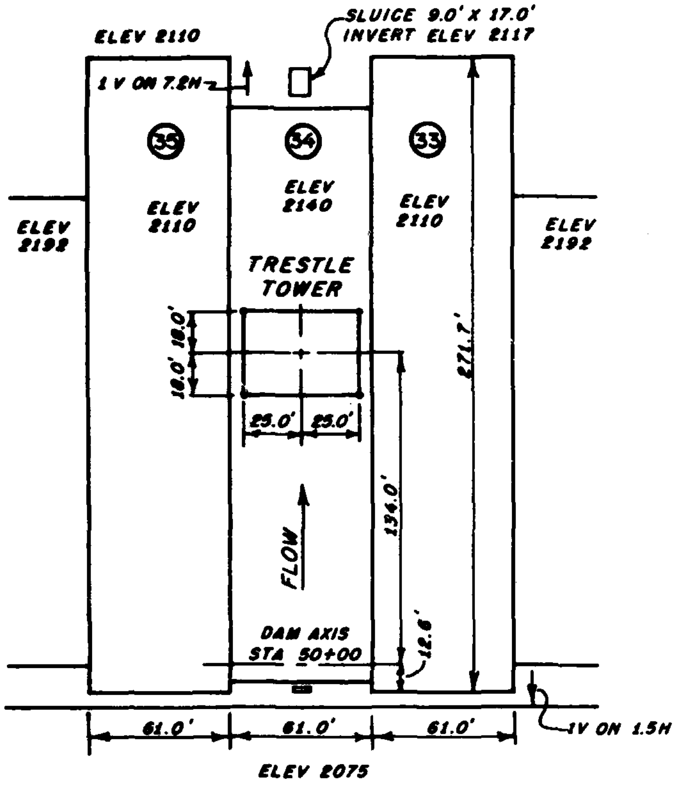

BANK VELOCITIES
SECOND-STEP, SECOND-STAGE DIVERSION
RIVER DISCHARGE 82 000 CFS
LEGEND
6 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
MONOLITH 33 63 000 CFS
SLUICES 19 000 CFS
NOTES
1. BED CONFIGURATION BEFORE DIVERSION SHOWN
2. OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48.
PLAN F DIVERSION
(WITH MOVABLE BED)
PLAN
SCALE
0 50 100 200 300 FT
POWERHOUSE
1
2
3
4
5
6
7
8
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2148
MODEL LIMIT
STA 66+26.0
TAILWATER CONTROL GATE
TAILWATER GAGE
2134.7
TEST AREA - SECTION A-A
SCALE
0 10 20 30 40 50 FT
7' 3000 TO 5000 LB DERRICK STONE
ELEV 2148
STIPPLED CONCRETE
GROUT
38.0'
ELEV 2110
MOVABLE BED
ELEV VARIES


BANK VELOCITIES
SECOND-STEP, SECOND-STAGE DIVERSION
RIVER DISCHARGE 109 000 CFS
LEGEND
7 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
MONOLITH 33 87 000 CFS
SLUICES 22 000 CFS
NOTES
1 BED CONFIGURATION BEFORE DIVERSION SHOWN
2 OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48
PLAN F DIVERSION
(WITH MOVABLE BED)
PLAN
SCALE
0 50 100 200 300 FT
POWERHOUSE
1 2 3 4 5 6 7 8
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2142
ELEV 2148
MODEL LIMIT
ELEV 2148
ELEV 2148
STA 66+26.0
TAILWATER CONTROL GATE
TAILWATER GAGE
2138.0
TEST AREA - SECTION A-A
SCALE
0 10 20 30 40 50 FT
1' 3000 TO 5000 LB DERRICK STONE
ELEV 2148
STIPPLED CONCRETE
GROUT
28.0'
ELEV 2110
MOVABLE BED
ELEV VARIES

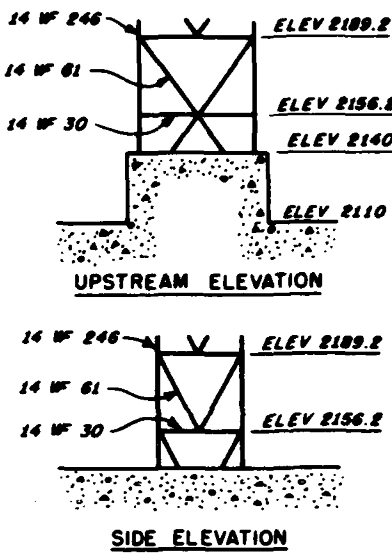

BANK VELOCITIES
THIRD-STAGE DIVERSION
RIVER DISCHARGE 20 000 CFS
LEGEND
3 VELOCITIES MEASURED IN FPS 4 FT ABOVE BOTTOM
OPERATING CONDITIONS
SLUICES 20 000 CFS
NOTE
1. BED CONFIGURATION BEFORE DIVERSION SHOWN.
2. OTHER DETAILS OF STRUCTURES AND CHANNEL SHOWN ON PLATE 48.
PLAN F DIVERSION
(WITH MOVABLE BED)
SCALE
0 50 100 200 300 FT
PLAN
POWERHOUSE
1 2 3 4 5 6 7 8
STA 53+88.0
ELEV 2073
ELEV 2142
ELEV 2148
MODEL LIMIT
STA 66+26.0
TAILWATER CONTROL GATE
TAILWATER GAGE
2125.1
7' 3000 TO 5000 LB DERRICK STONE
ELEV 2148
STIPPLED CONCRETE
GROUT
28.0'
ELEV 2110
MOVABLE BED
ELEV VARIES
TEST AREA – SECTION A-A
SCALE
0 10 20 30 40 50 FT



BANK VELOCITIES
THIRD-STAGE DIVERSION
RIVER DISCHARGE 30 000 CFS

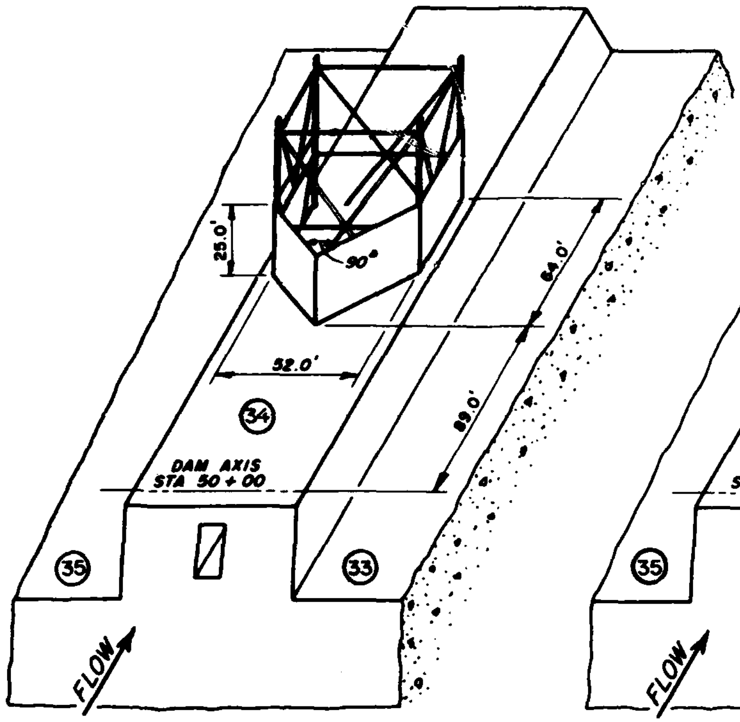


PLAN

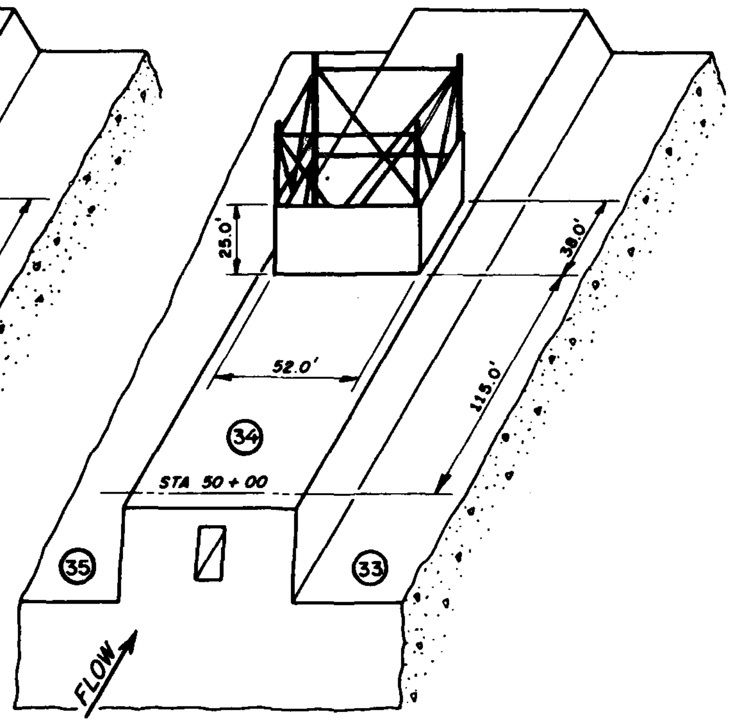


TRESTLE TOWER

LOW MONOLITHS 33 TO 35
AND
TEMPORARY TRESTLE TOWER



90-DEGREE DEFLECTOR

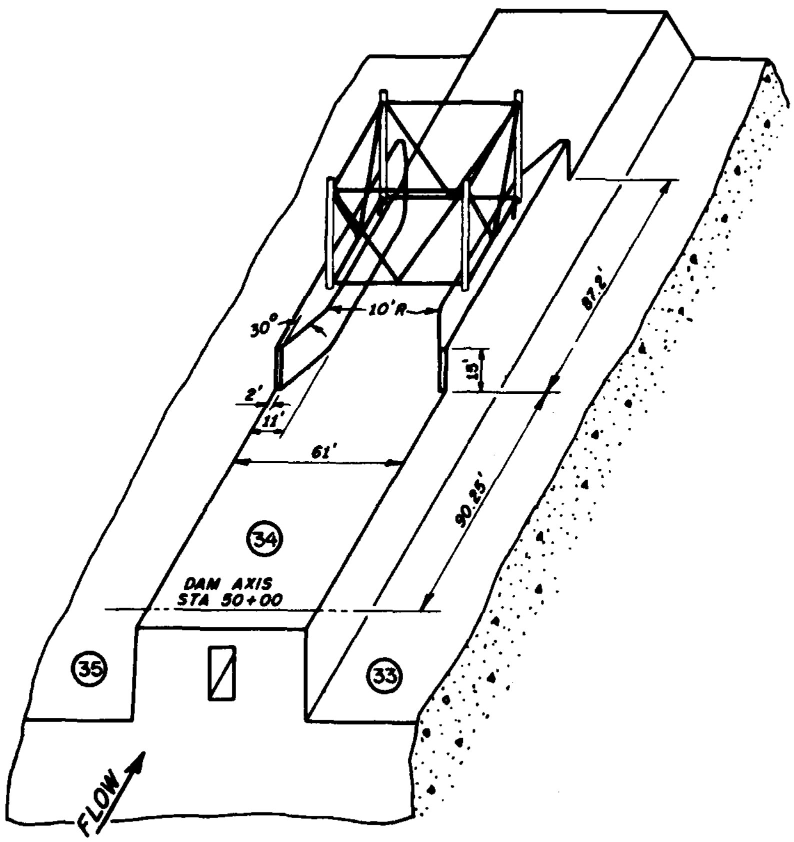


180-DEGREE DEFLECTOR

NOT TO SCALE

DEBRIS DEFLECTORS

LOW MONOLITHS 33 TO 35



CONTRACTOR'S DEBRIS DEFLECTOR

APPENDIX A

REDUCTION OF ABSORBED NITROGEN

General

1. Flow over the Libby spillway or through the regulating sluices will follow the concrete outline of the structure and plunge to the bottom of the stilling basin. Near the bottom, pressures of more than 40 feet of water exist which force nitrogen in the aerated flow into solution. Since all flow releases were to be made through the stilling basin until power units were completed, model studies were made in an attempt to develop modifications that would reduce nitrogen absorption in discharges up to 40,000 cfs. The studies were made with a 1:50-scale model described in paragraph 8 and shown in photograph 2 and plate 5.

2. Profile views that indicate the approximate zones of aeration in the stilling basin of the unmodified project are shown in photographs A-1 and A-2.

Effects of Deflectors and Flip Buckets

3. Prototype tests at Bonneville and Lower Monumental Dams had shown that flow deflectors installed on the spillways just below the minimum tailwater reduced the amount of nitrogen forced into solution during spill by diverting the aerated flow to the surface of the stilling basin. Since less pressure was exerted on the shallow aerated flow, less nitrogen was absorbed. Similar deflectors were tested on the Libby sluices.

4. The initial deflectors were 15-feet-long by 10-feet-wide horizontal surfaces at elevation 2115 that were connected to the sluice floors by 50-foot-radius curves. The deflectors did not prevent the flow from diverting into the basin. Tilting the deflectors upward 10 degrees and placing the invert at elevation 2110 increased surface flow in the basin with discharges as high as 20,000 cfs (photograph A-3). A discharge of 40,000 cfs plunged into the stilling basin and formed a hydraulic jump. Increasing the deflector length to

25 feet did not improve conditions with or without the 10-degree flip. Stepping the deflectors laterally with alternating 2-foot-wide flats (three at elevation 2115 and two at elevation 2110), reducing the entrance curve to a 25-foot-radius, and adding side walls to the deflectors were tried without success.

5. Flip buckets that would project the sluice flow over the stilling basin into relatively shallow water on the runout slope and reduce the amount of nitrogen that would be absorbed were tested (photograph A-4). With normal pool elevation 2459 and a 30-degree flip bucket with lip at elevation 2127.7, zones of impact were on the stilling basin runout slope (photographs A-5 to A-7). With minimum pool elevation 2287, impact was in the basin and entrained air was carried to the basin floor at the end sill. After several combinations of deflectors and flip buckets were tested, flip buckets were abandoned because potential damage to the runout slope and waves in the exit channel were considered to be too severe.

6. Following a model demonstration and conference on the nitrogen problem at Libby Dam, the conferees agreed that none of the plans were satisfactory.

Discharge 5,000 cfs, tailwater elev 2120.0

Discharge 10,000 cfs, tailwater elev 2121.5

Discharge 15,000 cfs, tailwater elev 2123.2

Discharge 20,000 cfs, tailwater elev 2124.5

Photograph A-1. Flow conditions in existing stilling basin, sluice discharge 5,000 to 20,000 cfs, pool elev 2459.0.

Discharge 25,000 cfs, tailwater elev 2125.7

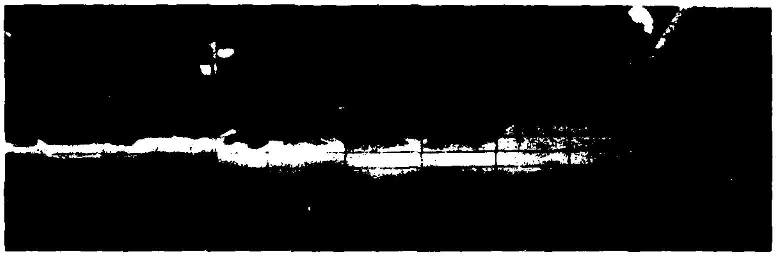
Discharge 30,000 cfs, tailwater elev 2126.7

Discharge 35,000 cfs, tailwater elev 2127.7

Discharge 40,000 cfs, tailwater elev 2128.6

Photograph A-2. Flow conditions in existing stilling basin, sluice discharge 25,000 to 40,000 cfs, pool elev 2459.0.

Discharge 10,000 cfs, tailwater elev 2121.5

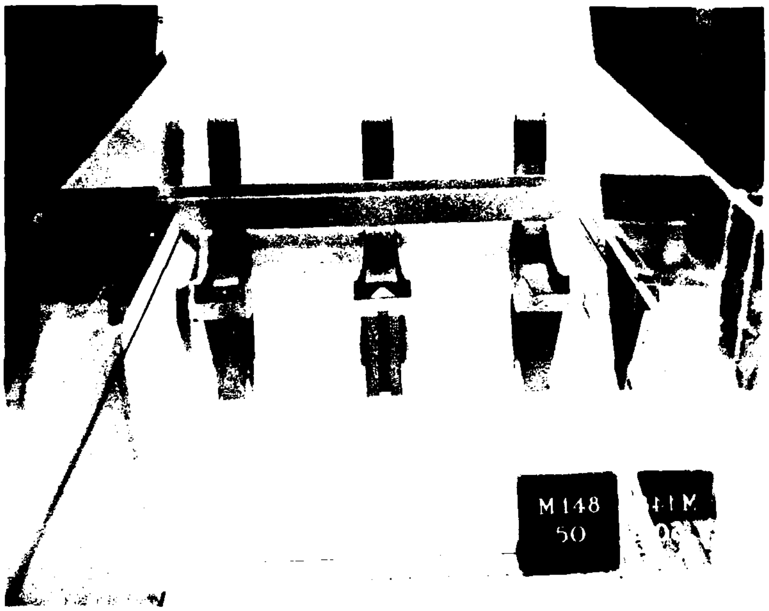


Discharge 20,000 cfs, tailwater elev 2124.5

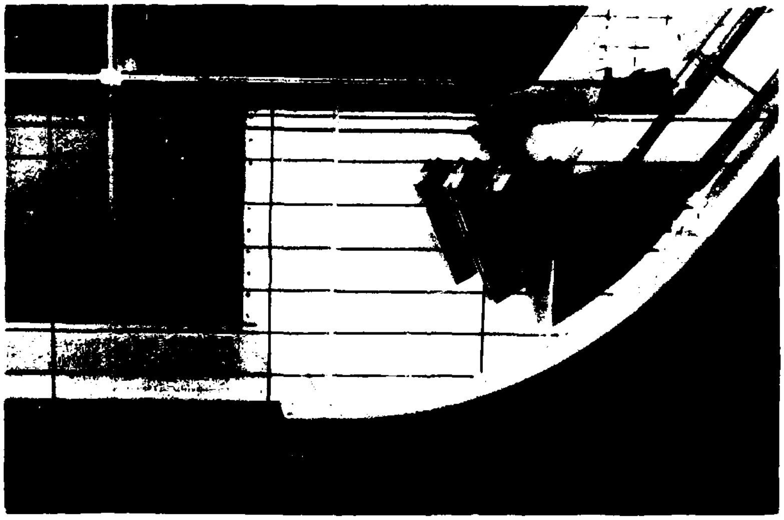
Discharge 10,000 cfs, simulated powerhouse discharge 10,000 cfs, tailwater elev 2124.5

Discharge 40,000 cfs, tailwater elev 2128.6

Photograph A-3. Flow conditions with sluice flow and plan B sluice deflectors (15 ft long at elev 2110.0 with 10-degree flip), pool elev 2459.0.

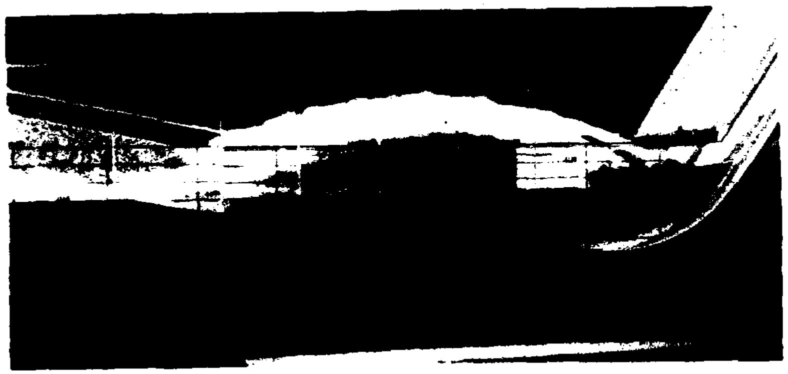


View from downstream



Side view

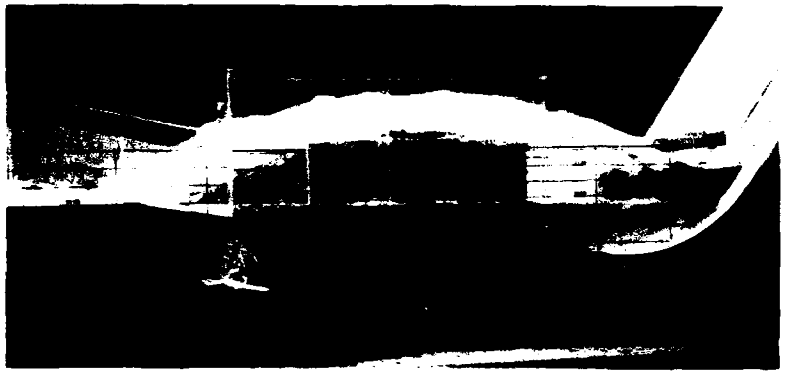
Photograph A-4. Plan E, 30-degree flip bucket at sluice outlets.



Side view

Plan view

Photograph A-5. Flow conditions with plan E flip buckets at sluice outlets. Discharge 10,000 cfs, tailwater elev 2121.5, pool elev 2459.0.



Side view

Plan view

Photograph A-6. Flow conditions with plan E flip buckets at sluice outlets. Discharge 20,000 cfs, tailwater elev 2124.5, pool elev 2459.0

Side view

Plan view

Photograph A-7. Flow conditions with plan E flip buckets at sluice outlets. Discharge 40,000 cfs, tailwater elev 2128.6, pool elev 2459.0.

DATE

FILMED

2 8