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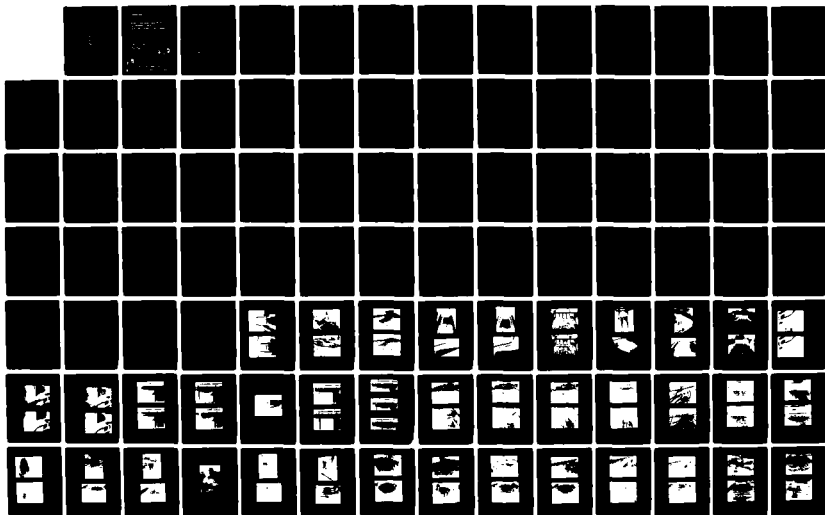
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**HYDRAULIC MODEL
INVESTIGATION**

TECHNICAL REPORT NO. 158-1

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Moose Creek Dam Outlet Works and Diversion Channel Chena River Lakes Project, Alaska

**SPONSORED BY
U. S. ARMY CORPS OF ENGINEERS
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**CONDUCTED BY
DIVISION HYDRAULIC LABORATORY
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NORTH PACIFIC DIVISION
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The Moose Creek project includes an earthfill dam, outlet works on the Chena River, and a diversion channel which diverts Chena River floodflows into the Tanana River. The report presents results of hydraulic model studies conducted to support final design of the outlet works and the diversion channel. Sixteen alternate designs were tested in development of the outlet works.		

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These designs were complicated due to the requirement to maintain fish and boat passage capability (to the maximum extent possible) during both gated and ungated operation of the structure and due to the lack of high-quality riprap required for channel protection downstream of the structure.

Fixed- and movable-bed models were used in the diversion channel studies. Primary emphasis was placed on evaluation of scour adjacent to bridge piers and on development of the overflow sill at the downstream end of the diversion channel where it enters the Tanana River. The overflow sill at the downstream end of the diversion channel was developed through the use of a 1:12-scale movable-bed model. *+*

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PREFACE

The Chena River Lakes project, authorized by the Flood Control Act of 1965, Public Law 90-483, 90th Congress (S-3710), provided for the construction of a dam, reservoir, and outlet works on the Chena River for flood control, recreation, and fish and wildlife enhancement. Plans were subsequently revised to eliminate the permanent reservoir behind the dam and the proposed pilot exit channel at the Tanana River and to replace them with a cleared floodway and diversion channel. This report is concerned with hydraulic model studies of the Moose Creek Dam outlet works and diversion channel.

Hydraulic model studies for the outlet works and the diversion channel were authorized on 7 August 1973 and 24 November 1976, respectively, by the Office of Chief of Engineers at the request of the U.S. Army Engineer District, Alaska (NPA). The studies were conducted in the North Pacific Division (NPD) Hydraulic Laboratory, Bonneville, Oregon, during the period March 1974 - April 1980 under the direction of Messrs. P. M. Smith, Director of the Laboratory, and A. J. Chanda and R. L. Johnson, Chiefs of the Hydraulics Branch. The tests were conducted under the supervision of Mr. B. B. Bradfield, engineer in charge of the model group. This report was prepared by the Seattle District Hydraulics Section.

NPA was furnished with preliminary data from the model tests during the course of the studies. Numerous personnel from NPD, NPA, and the Waterways Experiment Station (WES) visited the Laboratory to observe model tests and discuss test results. The outlet works model demonstrations and conferences were conducted at the Laboratory for the Chena Interagency Fish Passage Technical Committee to obtain advice and concurrence on flow conditions affecting fish passage at the project.

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

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

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
miles	1.609344	kilometres
feet per second	0.3048	metres per second
cubic feet per second	0.028317	cubic metres per second
pounds (mass)	0.4535924	kilograms
inches	2.54	centimetres

MOOSE CREEK DAM OUTLET WORKS AND DIVERSION CHANNEL
CHENA RIVER LAKES PROJECT, ALASKA
Hydraulic Model Investigation
PART I: INTRODUCTION

The Project

1. Moose Creek Dam is located approximately 17 miles* east of Fairbanks, Alaska, on the Chena River (figure 1). The project layout is shown on plate 1. The project, a part of the Chena River Lakes project, is primarily for flood control for the city of Fairbanks. Recreation and fish and wildlife enhancement benefits are also attributable to the project. The project site lies near the northern edge of the broad, flat valley of the Tanana River, which is a tributary of the Yukon River.

2. Moose Creek Dam, an earthfill structure with an average height of 30 feet, extends 7.1 miles across the Chena River valley. An outlet structure controls flow in the Chena River to prevent inundation of the flood plain downstream of the dam. Floodflows are diverted through the floodway to the south end of the dam and into the Tanana River through a diversion channel, thus bypassing the city of Fairbanks.

3. The outlet structure consists of four 25-foot-wide gated bays and is located adjacent to the left bank of the existing river channel at dam station 406+88. Approach and exit channels divert the river flow from its natural course upstream of the dam, through the outlet structure, and back into the river downstream of the dam. Floodflows are regulated with vertical lift gates in each of the four bays.

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

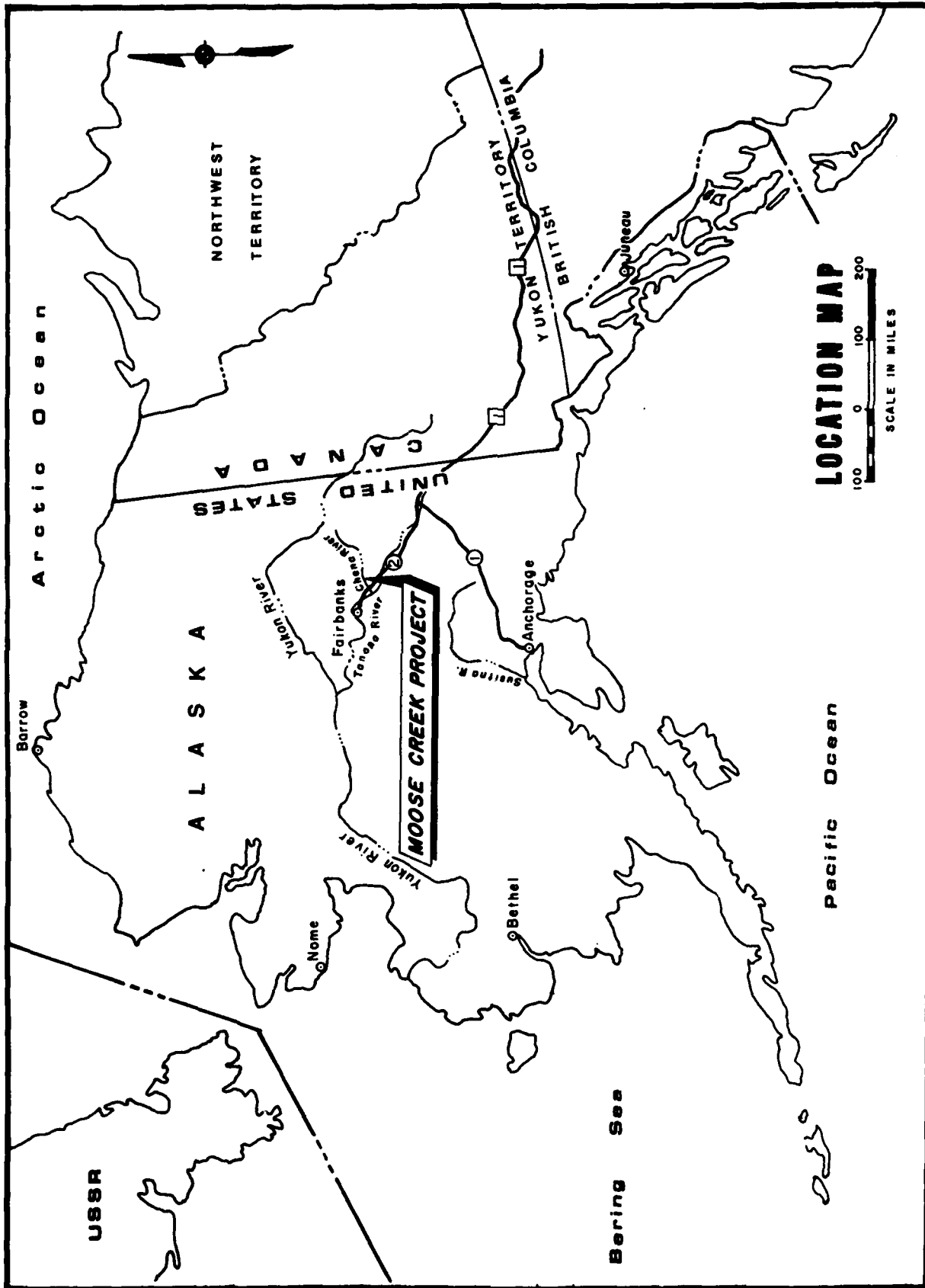


Figure 1

Maximum design discharge is 9,000 cfs with pool elevation 523.1.* The maximum opening at the gates is 18 feet above the approach apron (elevation 480.0) to provide adequate clearance for small boat passage during all ungated flows. The piers separating the bays have sloping ice-breaker noses on the three interior piers and vertical half-round noses on the two exterior piers. A shallow hydraulic-jump-type stilling basin is utilized to dissipate the energy of gated flows. The basin has a short horizontal apron with a double row of baffles and a sloping end sill. Narrow fishway slots are provided adjacent to the right and left abutments of the outlet structure, and a fish ladder is proposed near the left fishway entrance.

4. The diversion channel is located near the south end of the dam and floodway, extending approximately 2.8 miles from Moose Cr Bluff to the Tanana River. The channel is trapezoidal in cross section, has a 2,000-foot base width and is excavated to elevation 2. The channel narrows to a minimum width of 894 feet where twin bridges of a relocated section of the divided Richardson Highway cross the channel. The Eielson branch of the Alaskan Railroad crosses the diversion channel 1,090 feet downstream from the highway crossing. At the downstream end of the channel, a fixed overflow sill controls water level within the channel and will prevent backflow into the reservoir area from the Tanana River. The channel invert will be planted in grass maintained to a height of 12 inches. Clearing and grubbing of wooded areas adjacent to the channel will be kept to a minimum. Floodway flow will be constricted between Moose Creek Bluff and the dam, but flows approaching the 10-year flood and higher will overtop the channel banks downstream of the bluff and inundate the surrounding wooded area. Flows approaching the standard project flood (SPF) and greater will overtop the highway and railroad embankments to the east.

* Elevations referenced to MSL datum.

Purpose of the Model Studies

5. Hydraulic model tests of the outlet works were conducted primarily to develop a satisfactory stilling basin design and to determine whether the approach and exit channels to the outlet structure would be adequate to maintain a satisfactory water surface gradient through the reach. Flow conditions affecting fish passage and small-craft navigation through the outlet structure were studied, as was the flow regime in the river channel downstream from the project. The model was also used to observe debris passage in the approach channel and through the outlet works.

6. The model studies of the diversion channel were performed to obtain the data necessary to verify hydraulic design criteria that could not be reliably computed and to determine the overall hydraulic effectiveness of the channel. Design criteria that were studied included hydraulic losses at channel constrictions and expansions, water surface profiles, and division of flow between the diversion channel and overtopped embankments. Movable-bed models were required to provide data on which to base riprap requirements for bridge abutments and piers.

PART II: THE MODELS

Description

7. Two models were used to study the proposed outlet works and channel modifications as follows:

a. A 1:20-scale model simulating one 25-foot-wide outlet bay and half of each adjacent pier (photograph 1) was used to study the stilling basin design and determine the outlet discharge rating relationship. The model reproduced 163.5 feet of the approach channel upstream from the dam axis and 228 feet of the exit channel downstream from the axis and was constructed of waterproofed wood. The vertical-lift gate was wooden with an acrylic plastic lip. A 50-foot-long section of loose crushed rock simulating the 1-foot-diameter (prototype) riprap was incorporated in the model immediately downstream from the stilling basin.

b. A 1:40-scale comprehensive model (photograph 2 and plate 2) reproducing the Chena River from about 3,865 feet downstream of the dam to about 4,760 feet upstream of the dam and including the dam and outlet structure was used to evaluate fish and navigation passage at the project and to aid in design of outlet structure approach and exit channels. Fish ladder flows were drawn from a separate source and added at the ladder entrance. Model topography, excavated channels, and the dam were constructed of concrete molded between sheet metal templates to conform to field surveys and design plans. The outlet structure was constructed of waterproofed wood and plastic. Channel and overbank roughnesses were adjusted by adding stippled concrete and gravels cemented-in-place to reproduce prototype water surface elevations.

8. Four models were constructed to study the diversion channel as follows:

a. A comprehensive model built to a distorted scale of 1:100 horizontal (H) and 1:25 vertical (V) reproduced an area extending approximately 1 mile upstream from Moose Creek Dam, paralleling the dam between station 174+31 and ex-station 55+00, and extending approximately 2,000 feet into the Tanana River (plate 3 and photograph 3). The following primary features were included in the model: the upstream face of the dam; 3,000 feet of the approach floodway; the diversion channel, bridges, and embankments of the Richardson Highway and the Alaska Railroad; an overflow sill into the Tanana River; a portion of the Tanana River; and, in the final model studies, a raised access road to the overflow sill. Sufficient overbank to the left of the diversion channel was included to allow reproduction of floodway flows beyond the channel limits. The model was constructed of concrete and sand/cement mortar; sheet metal templates which conformed to field surveys were used to mold the floodway and river channels. The bridges and piers were made of wood, and the overflow sill was constructed of white plastic. Required roughness in the diversion channel was obtained with grout stippling and plastic boxwood plants (photograph 4). Required roughness in the Tanana River channel was obtained with large cobblestones and heavy grout stippling. Dense underbrush and trees up to 10 feet high in uncleared areas were simulated by plastic-coated horsehair mats 2 inches thick; trees 20 feet high were simulated by 1/4-inch hardware cloth.

b. Several types of overflow sills and drop structures were tested in a 1:12-scale movable-bed sectional model (plate 4). The model simulated a 19.8-foot section of the 2,000-foot-long sill. The movable bed extended approximately 50 feet upstream and 100 feet downstream from the sill. The bed material was medium to fine sand with an average grain size of 0.43 mm (5.2 mm in the prototype) which did not fully simulate the prototype size and gradation to scale; however, prototype conditions were approximated well enough for the results to be adequate for design purposes. Tests were of sufficient time duration for the movable bed to reach an approximately stable condition.

c. A sill built to the scale of the comprehensive model (1:100H/1:25V) would have created incorrect local flow and scour patterns downstream of the sill where qualitative observations were desired. The sill developed in the 1:12-scale model was tested in a 1:100H/1:25V-scale model in a movable-bed flume where the basin of the sill was distorted horizontally to cause scour to the correct depth and pattern as previously determined in the 1:12-scale movable-bed model. This distorted version of the sill and basin was then incorporated in the comprehensive models so that results from later comprehensive model tests could be properly interpreted.

d. A 1:15-scale movable-bed model was used to study scour around highway and railroad bridge piers and highway bridge abutments. The model size provided fully turbulent flow and adequate ratios of flow depth to material size for estimating scour depth in the prototype. The models were located in a 12-foot-wide by 38-foot-long flume. Bed material used in the majority of the studies was medium to fine sand with an average grain size of 0.43 mm (6.45 mm in the prototype); this material simulated the coarser 40 percent of the prototype size and gradation to scale. Limited studies were made using fine sand with an average grain size of 0.28 mm (4.20 mm in the prototype) and a coarse sand with an average grain size of 3.30 mm (49.50 mm in the prototype).

9. Water used in the operation of the models was supplied by pumps in a recirculating system. Tailwater elevations were controlled with adjustable tailgates. Piezometers were located in the lip of the vertical lift gate and two stilling basin baffle blocks of the outlet sectional model (plate 5). Standard laboratory procedures were used to measure discharge, water surface elevation, velocity, and current direction in the models.

Model Similitude

10. Accepted equations of hydraulic similitude based upon Froudian relationships were used to determine the mathematical ratios between the dimensions and hydraulic quantities of the models and the prototype. The movable-bed models were constructed to undistorted scales that provided fully rough turbulent flow and adequate ratios of flow depth to material size for estimating depth and extent of scour in the prototype.

PART III: OUTLET STRUCTURE AND STILLING BASIN

Original Design

11. The original design is shown on plate 6 and photograph 1. The 25-foot-wide bays were separated by 8-foot-wide piers with both approach and exit channel inverts at elevation 480. The interior piers had icebreaker noses skewed 30 degrees to the flow axis and sloped 45 degrees vertically. The vertical-lift control gates had a maximum opening of 18 feet above the approach invert. The 21-inch-wide gate lip consisted of a 2-inch horizontal seal plate having a 45-degree upstream face (plate 5). Downstream from the gate, the invert sloped downward at a 1V:14H slope to a stilling basin with an invert elevation of 479. The stilling basin was horizontal and extended 54 feet from the gate to the downstream edge of the end sill. The 2-foot-high end sill had a 1V:2H upward slope and rose to elevation 481--1 foot above the channel invert. A single row of six baffle blocks, each 2 feet wide by 3.4 feet long by 2.4 feet high, was located 24.5 feet downstream from the gate.

12. The criteria used in evaluation of the design were as follows:

a. The hydraulic jump resulting from controlled (gated) operation was to be entirely contained within the stilling basin.

b. The average pressure on the baffle blocks was to be no lower than -15 feet of water.

c. Maximum velocities in the channel immediately downstream from the basin were to be no greater than 5 fps due to the lack of high-quality riprap which would be required for channel protection with higher velocities.

d. Flow conditions during both gated and ungated operation had to be such that fish and boat passage capability would be maintained to the maximum extent possible.

e. The control gates were to remain stable under all operating conditions. With the maximum design discharge of 9,000 cfs, the criteria were applied to both a normal operating condition (four operational bays and normal tailwater elevation 494.4 feet) and an extreme operating condition (three operational bays and a tailwater 2 feet lower than normal).

Following are the operating conditions which were studied in the model:

<u>Outlet Discharge cfs</u>	<u>No. of Bays in Operation</u>	<u>Discharge per Bay cfs</u>	<u>Pool Elevation feet</u>	<u>Tailwater Elevation feet</u>
2,000	4	500	Ungated flow	487.1
2,000	4	500	513.0	487.1
2,000	4	500	523.1	487.1
9,000	4	2,250	Ungated flow	494.4
9,000	4	2,250	523.1	494.4
9,000	4	2,250	523.1	492.4
9,000	3	3,000	523.1	494.4
9,000	3	3,000	523.1	493.4
9,000	3	3,000	523.1	492.4

13. Initial tests were conducted with the normal operating condition--four operational bays and normal tailwater. With the maximum design discharge of 9,000 cfs and pool elevation 523.1, at least 50 percent of the hydraulic jump was not contained in the stilling basin. Channel bottom velocities 5 feet downstream from the end sill were 10 to 12 fps. Since such conditions did not meet the design criteria, tests on the original design were discontinued.

14. Sixteen alternate designs--with varying apron elevation, basin length, and baffle block configuration--were tested in an effort to obtain good energy dissipation of gated discharges. With maximum gated flow conditions, water entered the basin with a Froude number of 6 to 7 indicating (1) that the elevation of the stilling basin floor should be from 1 to 2 feet deeper than in the original design and (2) that with a corresponding basin length to conjugate depth ratio of 4:1 the basin should be about 20 percent longer than that of the original design. All subsequent basin designs were deepened to either elevation 477 or 478. Plates 6 through 10 show the major revisions to the basin in the development of the final design. All plans subsequent to the original design had a 50-foot-long section of the invert downstream from the end sill covered with a 2.5-foot-thick blanket of 1-foot-diameter rock. Late in the development stage of the basin design the design tailwater was lowered as a result of a downstream channel excavation modification. The tests indicated that increased basin length alone was not adequate to achieve satisfactory energy dissipation and that baffle blocks were also required. Test results for all 16 plans are summarized in tables A and B.

Final Design

15. The Plan 14 (plate 8) outlet structure and stilling basin as developed in the 1:20-scale model was originally proposed for the final design and included in the 1:40-scale comprehensive model. Features of the Plan 14 design which are significantly different from the original design are summarized below:

- a. The basin length between the control gate and the downstream face of the end sill was increased from 54 to 75 feet.
- b. The baffle blocks were increased in size and two rows of blocks were used instead of one row.
- c. The basin floor was lowered 2 feet to elevation 477.0.

d. The slope on the upstream face of the end sill was decreased to 1V:3H.

e. A 50-foot-long riprap section was added immediately downstream from the basin in the invert of the channel.

Structural requirements later necessitated two additional changes to the Plan 14 design which were subsequently tested in the 1:20-scale model. Plan 15 (plate 9) was identical to Plan 14 except that the stoplog slots were moved 3 feet downstream, their width was increased from 2.0 to 2.9 feet, and the 2.5-degree sloping invert downstream from the gate seal plate was extended 1 foot beyond the slots. The revised final design--Plan 16 (plate 10)--eliminated the slope downstream from the gates to provide a flat surface for the stoplog seals.

16. Tailwater elevations for basin Plans 1 through 14 were set for the original-design (Plan A) outlet channel and had a normal tailwater elevation of 494.4 for an outlet discharge of 9,000 cfs. During comprehensive model testing the Plan B (final design) outlet channel was developed which created a normal basin tailwater elevation of 493.4 for an outlet discharge of 9,000 cfs. The Plan 15 and 16 basins were tested only with (1) an outlet discharge of 9,000 cfs with both three and four bays operating and a normal tailwater elevation of 493.4 and (2) a 2-foot loss of tailwater (elevation 491.4). Additionally, Plans 15 and 16 were tested with a tailwater elevation of 494.4 for comparison with the Plan 14 basin. Table B summarizes the Plan 14 tests with 9,000-cfs outlet discharges and the Plan 15 and 16 tests.

17. The Plan 14 design provided a good hydraulic jump for the full range of discharges and tailwater elevations tested. The jump was completed within the basin, the flow was evenly distributed in the bay and over the end sill, and no return flow was observed. Velocities 5 feet downstream of the sill and 1 foot above the riprap were 5 fps or less for all discharges. The minimum pressure on the test baffle was -14 feet of water; this occurred with a discharge of 3,000 cfs per bay

(three bays operating) and tailwater which was 2 feet below normal. A minimum pressure of -4 feet of water occurred with 2,250 cfs per bay (four bays operating) and normal tailwater. Waves were generated by reflections from the upstream pier noses and converged in the center of the bay; however, wave heights within the structure were negligible. The pressure fluctuations on the upstream 45-degree sloping face at the bottom of the vertical-lift gate were less than 0.2 feet of water in amplitude with a period of approximately 4.5 seconds. Such conditions indicated that hydraulic vibrations of the gate would be insignificant. With an outlet discharge of 2,000 cfs (500 cfs per bay) the velocity downstream from the structure averaged 3 fps. Photograph 5 shows the extreme operating condition--3,000 cfs per bay, and three bays operating with a tailwater elevation 2 feet below normal (392.4).

18. The Plan 15 basin design provided satisfactory energy dissipation within the basin for the conditions with (1) a 9,000-cfs outlet discharge and normal tailwater (elevation 393.4) with three or four gates operating, and (2) a 9,000-cfs outlet discharge with tailwater 2 feet below normal and four gates operating (table B). Bottom velocities above the riprap (location C, table B) did not exceed 5 fps, and pressure on the test baffles was well above -15 feet of water. However, with the extreme operating condition of 3,000 cfs per bay (three bays operating) and a tailwater 2 feet below normal (elevation 391.4) the jump was forced downstream. Bottom velocities over the riprap were 7 to 8 fps, and pressures on test baffles were below -15 feet of water at three locations indicating that cavitation could be expected with that operating condition. Photograph 6 shows flow conditions for the extreme operating condition.

19. The hydraulic jump occurring with the flat entrance invert of Plan 16 was essentially the same as that with Plan 15, and satisfactory energy dissipation occurred within the basin with normal tailwater conditions. A larger portion of the jet passed over the baffles, and generally higher velocities occurred downstream of the structure.

However, bottom velocities above the riprap did not exceed 5 fps, and pressures on the test baffles were well above -15 feet of water with normal tailwater. With the tailwater elevation 2 feet below normal, velocities above the riprap exceeded 5 fps with discharges of 2,250 and 3,000 cfs per bay and pressures at the test baffle were below -15 feet of water at some piezometer locations which indicated that cavitation could be expected.

Discharge Rating

20. A discharge rating was made in the single-bay model with the Plan 14 stilling basin. Head-discharge relationships for four randomly picked openings of the vertical-lift gate are shown on plate 11 and in table C. With normal tailwater elevations (Plan B outlet channel), the gate bottom was submerged at all flows. With the gate openings observed, the water surface at the downstream face of the gate was 5.2 to 7.5 feet above the gate lip. The following equation was used to compute the gate discharge coefficient:

$$Q = C A \sqrt{2g (d_u + h_v - d_d)}$$

where

Q = Discharge per bay, in cfs

C = Coefficient of discharge

A = Area of gate opening, feet²

g = Acceleration of gravity, feet/sec²

d_u = Depth above invert upstream of gate, feet

h_v = Velocity head upstream of gate, feet

d_d = Depth above invert downstream from jump, feet

Discharge coefficients varied with gate opening and ranged from 0.809 to 0.845.

PART IV: OUTLET WORKS COMPREHENSIVE MODEL TESTS

21. The 1:40-scale comprehensive model included 1.7 miles of the Chena River with adjacent flood plain and the four-bay outlet structure developed with the 1:20-scale model. The outlet structure incorporated the 5-foot-wide fishways along the outside of both exterior piers and the entrance to the proposed fish ladder located near the left fishway entrance. The fish ladder was not modeled but fish ladder flows were simulated. The model was used to determine (1) flow conditions affecting fish passage and navigation through the outlet structure, (2) adequacy of the transitions and outlet channel, and (3) hydraulic performance of the stilling basin developed in the 1:20-scale model.

Verification

22. The 1:40-scale model of the existing river was verified by adjusting the roughness of the river bed until the prototype water surface elevations were reproduced in the model with acceptable accuracy. Prototype water surface elevations consisted of both observed data and computed elevations based on the observed data. Water surface elevations were compared at eight gage locations for six river discharges ranging from 1,120 to 9,000 cfs. The limits of the model study and the gage locations are shown on plate 12. Stippled concrete stucco and 1-inch-minus gravel were used for roughness in the model. The model/prototype agreement was good, varying no more than 0.2 foot. Correlation of the model/prototype data is shown on plate 13.

Original Design

Outlet Channel, Outlet Structure, and Fishway Entrance

23. The Plan A outlet channel (plate 12) was alined so that it would be normal to the outlet structure from 885 feet upstream to 1,350 feet downstream of the dam axis. The channel was trapezoidal

with a bottom width of 80 feet and 1V:2H side slopes, and was entirely riprapped with 1-foot stone. The Plan A outlet structure (photograph 7) is based on the Plan 14 basin that was developed in the 1:20-scale model. The structure had four 25-foot-wide by 18-foot-high gated bays between 8-foot-wide piers and a shallow baffled stilling basin for each bay. The upstream end of the three interior piers had icebreaker noses, and the two exterior piers had rounded vertical noses. All piers extended downstream 75 feet from the vertical-lift gates the full length of the stilling basin. The stilling basin was 3 feet deep and had two rows of 2.1-foot-wide by 4-foot-high baffle blocks with seven and six per row in each bay. The end sill sloped 1V:3H to elevation 451--1 foot higher than the channel invert at the outlet structure. A 5-foot-wide fishway was placed along the outside of each exterior pier, and the left fishway entrance included the fish ladder entrance.

24. Model observations revealed that the Plan A outlet channel was too short to develop the energy losses that occurred in the natural river. Consequently, design water surface elevations did not occur at the outlet structure and flow conditions that are required for navigation and fish passage were not obtained. The Plan A outlet structure was modified by shortening the downstream ends of the interior piers by 21 feet (Plan B and photograph 8). The shorter piers, which would be more economical than those of the original design and were also thought to improve fish movement through the stilling basin, did not adversely affect energy dissipation in the stilling basin.

Final Design

Outlet Channel

25. The Plan B outlet channel (plates 12 and 14 through 17) was selected for final design. The upstream approach channel generally followed the route of an old meander loop of the river and closely approximated the physical characteristics of the natural river channel, thus creating the required energy losses. The downstream exit channel

was subsequently realigned with a slight curvature to the right (plates 18 and 19) to simulate what was considered to be an estimate of a stable condition expected to develop in the prototype. The channel had riprap protection only on the side slopes in critical areas and in the channel transitions adjacent to the structure.

26. Correlation of the model water surface elevations and the prototype (observed and computed) water surface elevations is shown on plate 20 and table D. Prototype data were available either from observations in existing river reaches or from adjustment of those data. Computed water surface elevations were derived by analytical techniques for flow in the proposed outlet channel and structure. The agreement between the model and observed/computed data was good, varying no more than 0.3 foot.

27. Plates 21 through 30 show velocity contours measured 1 to 1.5 feet above the bottom of the Plan B outlet channel for ungated discharges of 1,630, 7,200, and 9,000 cfs. Plate 31 shows velocity cross sections taken at 0.6-depth at each of the four critical impact areas in the Plan B channel. The greatest flow impact on the upstream channel banks for two higher discharges occurred along the right bank from station 33+80 to 42+20 (outlet channel sections 29 to 37), the left bank from station 19+80 to 32+70 (sections 16 to 28), and the right bank from station 11+97 to 17+80 (sections 8 to 14). These high-flow impact areas extended farther downstream by 500, 400, and 300 feet, respectively, than each of the three riprapped sections (plates 14 and 15). In the downstream outlet channel, the right bank riprap (stations 0+00 to 3+22, plate 16) was not in an impact area and would not be required. The higher velocities in this section of the outlet channel occurred on the opposite side of the thalweg and indicated that the channel probably would not stabilize to the Plan B design in this area. With the estimated stable downstream river channel realignment (plates 18 and 19), the highest velocities occurred along the right bank from stations 62+00 to 66+00 and along the left bank from stations 56+00 to 58+00 (plate 30). Velocity cross sections for river stations

53+00, 57+85, and 63+00, at 0.6-depth are shown on plate 31. River verification data for stations 53+00 and 63+00 obtained prior to installing the Plan B channel and outlet structure are also shown on plate 31. Water surface elevations for the 9,000-cfs discharge were revised downward prior to the Plan B channel tests; therefore, the velocities shown for this discharge with Plan B are higher than those of the verification data. At river station 63+00, average velocities adjacent to the right bank increased 50 percent due to the upstream channel changes, while the flow distribution at station 53+00 remained about the same.

28. Surface flow conditions in the Plan B outlet channel with the ungated Plan C outlet structure and the downstream river channel are shown in photographs 9 through 12. Flow directions in the curved approach to the outlet structure (photograph 9) indicate that the greatest impact occurred along the right bank downstream from the rip-rapped (darker colored) side slope (paragraph 25). In the upstream transition, a large eddy occurred along the left side of the channel and a smaller eddy occurred along the right side with all discharges (photograph 10). The curved alinement of the approach channel produced unsymmetrical flow through the outlet structure with the smallest amount of flow--18 to 19 percent of the total flow--passing through the left bay (bay 1) and 27 to 29 percent passing through each of the right bays (bay 3 and 4). Table E shows the flow distribution through the outlet structure bays with the various conditions tested.

Outlet Structure and Fishway Entrance

29. The final design (Plan C) outlet structure is shown on plate 32 and photograph 13. Except for the exterior piers, transitions to the channel, and the fishway entrance, the design was essentially the same as Plan B. Both exterior piers were extended upstream 10 feet. Downstream the exterior piers were shortened 21 feet to match the length of the interior piers to provide fish passage from the stilling basin to the fishways, and the fishway face of each pier was tapered

45 degrees to reduce the width of the pier noses behind which eddies might form. A minor shift in the location of the stilling basin baffles was required due to the reshaping of the exterior piers. The transitions to the channel had vertical 40-foot-radius quadrants abutted by channel banks with 1V:2H side slopes. The fish ladder entrance was redesigned and located farther upstream in the left fishway. A 3-foot-high divider wall between the fishway entrance and the stilling basin (Plan C fishway, plate 32, and photograph 14) was tested at the request of the Chena Interagency Fish Passage Technical Committee.

30. The performance of the Plan C outlet structure is discussed in paragraphs 16 through 20. Further testing of the outlet structure in the comprehensive model was conducted primarily to evaluate adequacy of flow conditions with respect to fish passage.

31. Flow direction and velocities in the stilling basin and transitions for a range of ungated outlet discharges from 1,120 to 12,000 cfs are shown on plates 33 through 38. With all discharges, small eddies formed downstream of both exterior piers. Eddies also formed adjacent to the walls at the downstream quadrants with discharges of 6,080 cfs and greater. Due to the unsymmetrical approach flow to the outlet structure (paragraph 28), bottom velocities on the right side of both transitions exceeded 5 fps (considered maximum for riprap stability) with discharges of 4,000 cfs and greater (plates 35 through 38). Velocities were also greater than 2.5 fps in the right fishway. With ungated flow, no undesirable disturbance was created in the stilling basin by the baffles. Conditions resulting from a 1.3-foot drop in tailwater elevation at the structure--a possible effect of degradation in the downstream channel--are shown on plate 39 for a discharge of 9,000 cfs. Conditions with normal tailwater are shown on plate 37. The drop in tailwater caused a decrease in the size of the eddies along the banks of the upstream transition and caused a general increase in velocities of approximately 1 fps. The 3-foot-high divider wall in the Plan C fishway entrance did not affect flow conditions between the stilling basin and the entrance and did not eliminate the eddy downstream of the left exterior pier.

32. For gated outlet conditions, three different flow-distribution patterns were observed--uniform, ungated, and linearly varied. The percent of total flow in each bay for these gated conditions is shown in table E.

a. Uniform flow distribution. Plates 40 through 42 show velocities and flow directions for uniformly distributed gated outlet discharges of 2,000, 4,000, and 6,000 cfs, respectively, with pool elevation 510.0 (maximum for fish ladder operation). The Plan C structure provided a good hydraulic jump in the basin with the full range of discharges and tailwater elevations tested. Velocities 5 feet downstream from the end sill and 1 to 1.5 feet above the riprap did not exceed 4.8 fps. Flow conditions in the Plan B and C fishway entrances for discharges of 2,660, 4,000, and 6,080 cfs are shown in photographs 14 through 16, respectively. Velocities in the approach to the fish ladder were acceptable for fish attraction. As shown by the dye streaks in the photographs, the extended training wall of the Plan C fishway entrance had little effect on flow conditions at the entrance. In general, the velocities and flow patterns were the same with both Plan B and C entrances. No eddies were observed along the downstream abutment walls during gated flows.

b. Ungated flow distribution. Plates 43 through 45 show flow conditions in the stilling basin with outlet discharges of 2,000, 4,000, and 6,000 cfs, respectively, and discharge in each bay was adjusted to duplicate that which occurred with the outlet ungated. A 5 to 6 percent maximum deviation in flow from that which existed with uniform gate operation was achieved in the end bays without exceeding 5-fps bottom velocities on the right side of the downstream transition, yet producing acceptable velocities on the left side for fish attraction. The extended training wall of the Plan C fishway entrance had little effect on flow conditions at the entrance. The velocities and flow patterns were generally the same with both Plan B and C fishway entrances for identical outlet discharges.

c. Linearly varied distribution. The linearly varied pattern met the prescribed criteria of velocities of 2.5 to 3.5 fps at the end sill in bay 1 and bottom velocities not exceeding 5 fps over the riprap downstream from the basin. Velocities and flow directions obtained with discharges of 2,000, 4,000, and 6,000 cfs are shown on plates 46 through 48, respectively. For all gated flow conditions, no eddies existed along the downstream abutment walls.

33. Tests were made to determine the effect of proposed alining piles on floating timber debris approaching the ungated outlet structure. Three vertical piles were placed in the upstream transition--two 80 feet upstream of the outlet structure and 40 feet right and left of the channel centerline and one 170 feet upstream of the structure on the centerline. The majority of the debris was from 24 to 54 feet long with a maximum length of 85 feet and an average diameter of 11 inches. With a discharge of 7,200 cfs almost all of the debris floated adjacent to the right bank as it approached the outlet structure and was intercepted by the right pile. Debris seldom was intercepted by the center or left pile. When debris was first present, the right pile was effective in alining about 40 percent of the material so that it would pass through the outlet bays. Those pieces that failed to pass through were caught between the right pile and the right bank or between the pier noses of the bays. As soon as one or two pieces were caught, the following pieces were intercepted, and a jam was created. In tests without the alining piles, very little debris passed through the outlet structure. Most material caught on the pier noses of the two right bays.

PART V: DIVERSION CHANNEL

General

34. Four models were used in development of the diversion channel design (paragraph 8). All tests were accomplished with the following discharges: 15,030 (10-year flood), 40,500 (100-year flood), 74,000 (SPF), and 160,000 cfs (probable maximum flood, (PMF)). Concurrent flows in the Tanana River were 75,000, 104,000, 125,000, and 250,000 cfs, respectively. The model water surface profiles were calibrated to prototype data observed in the Tanana River and to computed depths in the diversion channel. Simulation of roughness elements is described in paragraph 8a. The computed depths in the diversion channel were based on a Manning roughness coefficient ("n") of 0.032 in the diversion channel (grass at a maintained height of about 12 inches) and 0.150 in the uncleared portion of the floodway. The 5-foot-high 45-degree training dikes upstream from the highway bridge abutments were too low to be effective. The flow followed the embankments and crossed over the dikes with all four discharges. The flow separated from the abutments just upstream of the upstream bridge, and most of the flow passed under the second spans from the abutments. Eddies formed in the spans at the abutments with the three largest flows. Some flow constriction also occurred at the railroad bridge indicating the need for training of the flow at the abutments during the SPF and PMF. Detailed evaluation of the losses at the bridges was not performed during verification.

Original Design

35. The original-design diversion channel is shown on plate 3. The overflow sill in the model was trapezoidal with a 1V:1H upstream face, a 1-foot-wide crest at elevation 506.65, and a 1V:2.15H downstream face. Photographs 3 and 4 show the model before and after adding roughness elements in the diversion channel.

36. Tests of the original design revealed that the discharge over the roadway embankments to the left of the diversion channel during the SPF and PMF was greater than predicted by the computed data.

37. The velocities 500 feet upstream from the overflow sill indicated that flow in the channel was uniformly distributed across the sill. The sill effectively controlled the outflow from the diversion channel, but energy dissipation downstream from the sill was not satisfactory.

38. With all four discharges a small, unmeasured portion of the flow left the channel on the left side downstream from the bridges and passed through the wooded overbank area and 15-Mile Slough to the Tanana River. Flow entering the Tanana River from the floodway and diversion channel had only a minor effect on Tanana River flow conditions with the 10-year flood; however, the effect increased as discharge increased. Water surface elevations at the dam created by a 50-percent blockage of the diversion channel at either the highway or the railroad bridge were observed with the PMF condition. The minimum and maximum freeboard along the dam embankment during such conditions were 1.2 and 1.5 feet, respectively.

Overflow Sill

39. Development of the overflow sill was accomplished in a 1:12-scale movable-bed model (plate 4 and paragraph 8b). Conditions simulated in the model tests included the PMF hydrograph with both a high and low tailwater; and the 10-year, 100-year, and SPF with high tailwater. The low tailwater used with the PMF hydrograph was obtained from comprehensive model tests with a minimum Tanana River discharge. The high tailwater simulated was obtained from the comprehensive model with coincident Tanana River and diversion channel discharge. Tests were of a sufficient time duration to permit approximate stabilization of the movable bed.

Initial Tests

40. Ten designs (plate 49) were tested in development of the overflow sill. The original design incorporated in the comprehensive model did not satisfactorily dissipate energy and was not tested in the 1:12-scale model. Three broad-crested weirs having variations of a 1V:6H runoff slope were initially tested, but all exhibited unstable hydraulic jumps and severe erosion at certain discharge/tailwater conditions. The broad-crested weirs considered initially in the study were abandoned in favor of a sharp-crested weir for economic as well as hydraulic reasons. The sharp-crested weirs tested consisted of Z-27 sheet piling and had concrete aprons placed along the downstream face to protect the piling from undermining. Three of the designs utilized an end sill on the apron to force a hydraulic jump on the apron, while the other four designs did not include an end sill but employed aprons of varied length, slope, and elevation.

41. All of the sheet pile sill designs having aprons without end sills produced more stable flow conditions downstream from the sill and were more effective in reducing scour than those with the broad-crested sills. Of the four designs tested, those with the downstream end of the apron at elevation 499.5 or lower produced a flow depth adequate to ensure that most of the energy dissipation occurred on the apron. After the PMF hydrograph, these structures had no more than 2 feet of scour at the cutoff wall and the maximum scour occurred to elevation 490 or 491 at a distance of 20 to 30 feet from the structure. An apron length of 32 feet was just as satisfactory as one of 40 feet. The 40-foot-long horizontal apron at elevation 502 produced supercritical flow off the end, and heavy scour occurred at the cutoff wall.

42. Tests were conducted for three designs consisting of sheet-pile-sill drop structures with aprons having end sills to force a hydraulic jump on the structure. The initial structure had two 20-foot-long basins with a 5.3-foot drop between them and a downstream

end sill at elevation 496.0. The double basin structure did not intercept the nappes of the higher discharges and severe scour occurred downstream. A single basin with a 40-foot-long floor at elevation 498.0 and a 2.25-foot-high end sill (Plan A) was slightly more effective than the 40-foot-long horizontal apron described in paragraph 41. After testing with the PMF hydrograph, less than 1 foot of the downstream face of the end sill was exposed.

Final Design

43. The Plan B basin (plate 50) was selected as the final design for further study in the comprehensive model. The basin had a 36-foot-long apron sloped from elevation 498.5 to 498.0 to provide drainage and a 2-foot-high end sill. The concrete apron cutoff extended downstream on a 1V:1H slope to elevation 492.0. The runout downstream from the end sill extended horizontally 13 feet and then sloped up to elevation 502.0. Photographs 17 and 18 show flow conditions during the PMF hydrograph with low tailwater. The structure was very effective in containing the hydraulic jump for flows up to the SPF, but the jump was not contained as well during the PMF. After testing with the PMF hydrograph, less than 1.5 feet of the downstream face of the end sill was exposed. The maximum erosion occurred at a distance of 25 feet downstream from the apron end sill where the bed eroded to elevation 493 (plates 51 and 52 and photograph 19).

44. With each of the sills that were tested, bed movement upstream from the structure was negligible for the 10- and 100-year floods and the SPF. Material moved downstream towards the sill with flows approaching the PMF, and deposited near the upstream face of the sill. Maximum deposition after testing of the final design Plan B basin with the PMF hydrograph was 2.5 feet with both high and low tailwater (plates 51 and 52).

Modification for Comprehensive Model

45. The Plan B sill and basin was to be included in the distorted scale comprehensive model for final design of the diversion channel. Flow over sills and scour are dependent on velocity (head) and depth; therefore, the patterns they create are primarily related to the vertical scale of a model. A sill built to the scale of the comprehensive model (1:100H/1:25V) would have created incorrect local flow conditions and scour patterns downstream of the sill where qualitative observations of scour were desired. Therefore, a 1:100H/1:25V-scale sill with the basin distorted horizontally to duplicate the scour depth and pattern occurring in the 1:12-scale model was developed in a movable-bed flume.

46. The 36-foot-long basin in the 1:100H/1:25V-scale model was too short to intercept the nappes of the SPF and PMF, thus resulting in abnormal scour downstream from the structure with either flow. Additional tests indicated that 90 feet of basin length (plate 53) would be required to intercept and turn the nappe for a satisfactory reproduction of scour. Flow conditions and scour profiles occurring with an elongated basin during the PMF hydrograph are shown on photographs 20 and 21. Scour profiles with low and high tailwater are shown on plates 54 and 55, respectively. Peak flow during the PMF hydrograph was maintained for 17 hours (in the prototype). Maximum scour occurred after 2 hours at which time the model bed had stabilized. Approximately 0.5 feet of the downstream face of the end sill was exposed, and maximum scour to elevation 493 had occurred at a distance of 132 feet downstream from the structure. Reproduction of scour in the horizontally distorted model compared closely with that obtained in the undistorted 1:12-scale model. A comparison of scour depth between the models is shown on table F. A comparison of the location of maximum scour downstream from the end sill is shown on plate 56. Due to the horizontal scale distortion, interpretation of scour location should be based on the 1:12-scale model tests.

Final Design

47. The final-design (Plan B) diversion channel is shown on plate 57. The originally designed 5-foot-high 45-degree wingwalls at the bridge abutments were replaced by 14.5-foot-high circular training dikes (plate 58). The silt blankets between the dam and the diversion channel were realigned to extend approximately 750 feet from the dam between station 174+00 and ex-station 100+00 to provide a smoother boundary for flow along the right side of the channel. A mound of earth adjacent to the left abutment of the railroad bridge was removed, and rock-protection for side slopes was eliminated along 1,500 feet of the left side of the channel downstream from the bridge. The sheet pile sill (plate 50) and a 36-foot-long stilling basin replaced the original trapezoidal sill and apron. To protect the sill structure from Tanana River flow, a dike was placed along 15-Mile Slough and approximately 2,900 feet into the Tanana River to deflect flow away from the structure. The top of the section of dike in the river tapered from elevation 515.0 adjacent to the bank to elevation 511.0 at the end. A dike parallel to the riverbank connected the diversion dike to the left abutment of the sill.

48. Plate 59 is a comparison of computed and model water surface profiles through the diversion channel. The maximum deviation between the model and computed data was 0.3 feet. General flow conditions in the diversion channel, floodway, and Tanana River are shown in photographs 22 through 25 and on plates 60 through 63 for the 10-year flood, 100-year flood, SPF, and PMF, respectively. Velocities in the channel averaged approximately 1 fps with the 10-year flood and 6 fps with the PMF. With each discharge large eddies occurred to the right and left of the diversion channel between the highway and railroad bridges and downstream of the railroad bridges. With all discharges a small amount of the flow entered the wooded overbank area to the left of the channel downstream of the bridges. Part of this flow passed to the Tanana River in 15-Mile Slough and the remainder reentered the diversion channel immediately upstream of the overflow sill.

49. The training dikes at the bridge abutments were investigated with arcs of 90 and 135 degrees at the highway bridges and 45 and 90 degrees at the railroad bridge. Flow conditions with 135-degree arcs at the highway bridges and 90-degree arcs at the railroad bridges are shown on plates 60 through 63. Flow conditions with 90-degree arcs at all bridges are shown on plates 64 through 67. Flow conditions with 90-degree arcs at the highway bridges and 45-degree arcs at the railroad bridge are shown on plates 68 and 69 (SPF and PMF only). Drawdown was reduced at the abutments with all discharges and arc lengths, and energy dissipation was reduced to a minimum. Velocities in the flow approaching the highway bridge adjacent to the 135-degree dikes ranged from 3.6 fps with the 10-year flood to 9.9 fps with the PMF. With 90-degree dikes, velocities ranged from 3.6 to 9.4 fps. At the railroad bridge the velocity range was from 3.4 to 8.5 fps with both the 45- and 90-degree dikes. The shorter arcs were as effective as the longer ones in reducing contraction at the abutments. The PMF flow overtopped the dikes at both bridges causing surface turbulence that extended beyond the adjacent bridge pier. Bottom flow remained aligned with the abutments and piers.

50. Surface flow conditions with SPF and PMF flows overtopping the highway and railroad embankments are shown in photograph 26. The extent of the overtopping and the resulting flow patterns is shown on plates 62 and 63. Approximately half of the flow returned to the diversion channel upstream of the railroad bridge with each discharge. Flow conditions were also observed during the SPF and PMF with a raised railroad embankment simulating a future addition of ballast that would prevent overtopping. SPF flow through the bridges with 135-degree dikes at the highway bridge and 90-degree dikes at the railroad bridge is shown on plate 70. With the SPF condition velocities at the highway bridge were increased approximately 9 percent at the left training dike, 5 percent at the right training dike, and 4 percent between piers 2 through 11. The water surface at gage 2 was 0.13 foot higher. During the PMF the water surface at gage 2 increased 1.30 feet; freeboard at the dam opposite gage 2 was slightly more than 1 foot.

51. Flow conditions at the overflow sill are shown in photographs 27 and 28 and on plates 71 through 74. The average flow velocity immediately upstream of the sill ranged from 2 fps with the 10-year flood to 7 fps with the PMF. The basin was effective in containing the hydraulic jump for flows up to the SPF, but the jump extended beyond the sill during the PMF particularly at the right end of the structure. A large eddy was present on the Tanana River between the left abutment of the overflow sill and the diversion dike.

52. To determine scour downstream from the overflow sill, the Tanana River channel adjacent to the sill was modeled in a movable-bed section 700 feet wide by 2,500 feet long. Tests were made with the PMF hydrograph with minimum tailwater and were of sufficient duration to permit approximate stabilization of the movable bed. The scour existing at the time of the SPF peak discharge (after 28 hours of the PMF hydrograph) and at the end of the test (after 38 hours) is shown on plate 75. At the SPF discharge, scour depth along the downstream face of the sill was less than 0.5 feet and maximum scour downstream from the sill was to elevation 496.5. Except at the scour hole adjacent to the left abutment, scour on the downstream face at the end of the the test was approximately 2.5 feet. The average maximum scour downstream from the sill was approximately to elevation 492. The scour depths corresponded closely with those observed in the 1:12-scale model. Due to the scale distortion, estimation of scour location in the prototype should be made from data observed in the 1:12-scale model (paragraph 46).

Bridge Pier and Abutment Scour

53. Tests to investigate scour around the piers and abutments of the Alaska Railroad and Richardson Highway bridge crossings over the diversion channel were conducted in a 1:15-scale movable-bed model (paragraph 8d). Gradation curves for the prototype backfill material at the bridge piers and for the material simulated in the model are shown on plate 76. Flow durations used in the tests were sufficient to permit approximate stabilization of general and local scour.

Railroad Bridge Pier Scour

54. The model layout and pier details are shown on plates 77 and 78, respectively. Results of scour around a single, isolated railroad pier aligned with the flow following the 100-year flood, SPF, and PMF are shown in table G, photographs 29 through 31, and plate 79. In all instances maximum scour occurred around the side corner of the diamond-shaped pier. Maximum scour was to elevations 498.5 and 496.5 with the 100-year flood and SPF, respectively. Small areas of the top of the pier footing (elevation 495.0) were exposed after testing with the PMF. Six additional tests were made with flow conditions which bracketed the approach velocities and flow depths of the PMF; test conditions and the resulting scour are listed in table H. Small portions of the pier footing were exposed with all but the lowest discharge. A heavy concentration of timber debris on the pier during the PMF (shown in photograph 32 at the base of the pier following the test) caused a larger exposure of the top of the pier footing and a greater extent of local scour; the results are shown in table G and photograph 33. Scour that occurred with a partial blockage of the channel at the bridge during a major flood (100 cfs/foot) is shown on table G and photograph 34. The width of the channel was reduced by one-third at a point immediately upstream of the pier and caused a 2.5-foot drop in head at the restriction at the beginning of the test. At the completion of the test, almost all of the top of the pier footing was exposed, but very little of the bed material along the sides of the footing was swept away. The test indicated that the potential for the occurrence of conditions conducive to channel blockage, resulting in undermining of the piers, depended largely upon the extent and location of the blockage material with respect to the piers.

Highway Bridge Pier Scour

55. Scour around the pier footings of the dual bridges crossing the diversion channel was studied in the model (plates 80 and 81). Three piers from each bridge were included in the model to simulate

flow constriction in the test channel; however, scour was only studied in detail at the two center piers. Tests were initially made with the piers aligned parallel to the direction of the flow. Scour occurring with the 100-year flood, SPF, and PMF is listed in table H and shown on plates 82 and 83 and in photographs 35 through 37. The maximum scour with the three flow conditions tested extended to elevations 497.7, and 497.0 at the upstream and downstream pier, respectively. The pier footings were not exposed during any of the tests.

56. Tests were also made with the flow approaching the highway bridge piers at an angle of 15 degrees. Scour resulting from the three floodflows is listed in table H and shown in photographs 38 through 40 and plates 84 and 85. At the upstream pier, scour reached elevation 497.0 with the 100-year flood and elevation 495.3 with the SPF. With the PMF, scour exposed the left half of the top of the upstream pier footing (elevation 495.0). At the downstream pier, scour reached elevation 498.0 with the 100-year flood and elevation 496.0 with the SPF. About one-third of the top left footing was exposed by the PMF. A heavy concentration of timber debris on the upstream piers produced serious undermining of the upstream pier footings with both the SPF and the PMF (table H, photographs 41 and 42, and plates 86 and 87); scour at the downstream piers did not extend below elevation 496.8 with either flood.

57. Tests were made to evaluate scour sensitivity to various channel and flow characteristics. Various characteristics evaluated are described below and test results are summarized in table H:

a. Conditions with a channel Manning roughness ("n") of 0.026 were compared with those having a roughness of 0.032. Scour with both conditions was almost identical.

b. Bed material with an average grain size of 0.25 mm (4.2 mm prototype) was tested and results compared with that in which the average grain size was 0.43 mm. Local scour with the finer grain size is

shown in photograph 43 and on plate 88. The test indicated that the variation of grain size within the range tested had negligible influence on local scour in the model.

c. A test was made to determine the effect of riprap on localized scour and the stability of the riprap. Rock protection simulating the gradation shown on plate 89 was placed around the center two highway piers. The riprap was placed over and around the pier footing from elevation 498.0 at the pier face to the base of the footing (elevation 492.0) at an angle that provided a minimum 2-foot thickness of protection at the top edges of the footing (photograph 44). The SPF exposed the riprap at both piers at the nose and along the left side to a maximum distance of 4 feet from the face. Maximum scour occurred along the left side of the downstream pier where riprap was exposed to elevation 497.0 (photograph 45 and plate 90). None of the exposed riprap was displaced by the flow.

d. A test was made to determine the affect that sediment-transport-inhibiting vegetation cover in the upstream approach would have on local scour. The upper portion of the sand-bed flume was "fixed" to within 38 feet (2.5 feet in the model) of the upstream piers to minimize sediment motion past the piers and to simulate a clear water scour condition. The SPF was repeated with the riprap protection described in paragraph 57c. The results of the test (photograph 46 and plate 91) were very similar to those which occurred with continuous sediment motion and indicate that equilibrium scour depths at the piers would be similar with and without sediment-transport-inhibiting vegetation.

58. During the study a 1:15-scale movable-bed model was constructed to develop data to supplement other generalized pier scour data observed by WES. The results of those tests are available at WES and documented in NPDEN-TE-L letter dated 7 August 1978, subject: Moose Creek Diversion Channel Highway Bridge Pier Scour.

Highway Bridge Abutment Scour

59. Movable-bed model studies were used to evaluate the local scour characteristics and adequacy of rock protection associated with the circular training dikes at the bridge abutments. The model is shown in photograph 47 and plate 92. Two riprap gradations were tested--type I around the bridge piers and type II on the upstream face of the dike. The riprap gradations are listed on figure 2. The bed material used in the model simulated the coarser 40 percent of the size and gradation of prototype bed material.

60. Scour resulting from the 100-year flood, SPF, and PMF is shown in photographs 48 through 50 and on plates 93 through 95. Slack flow occurred adjacent to the abutment between the bridges during all three discharges. Flow overtopped the training dike with the PMF and caused a surface disturbance that extended laterally into the channel beyond pier 14. At the end of the test, maximum scour depth at the toe of the training dike was 1 foot with the 100-year flood, 6 feet with the SPF, and 8 feet with the PMF. The type II riprap was not displaced by the 100-year flood or SPF flows. Movement of the riprap did occur with the PMF (plate 91) primarily because of flow overtopping the dike. Bed material was deposited along the abutment slope opposite the piers to a height of 4.7 feet during the 100-year flood, 7.0 feet during the SPF, and 6.0 feet during the PMF. Maximum local scour occurred as follows:

a. 100-Year Flood. Downstream pier 13 was exposed to elevation 497.0 (5.0 feet of scour).

b. SPF. Portions of the tops of the footings (elevation 495.0) of both piers 13 and downstream pier 14 were exposed (7.0 feet of scour).

c. PMF. The footings of both piers 13 were undermined to elevation 488.3 upstream (13.7 feet of scour) and to elevation 487.5 downstream (14.5 feet of scour).

TYPE I RIPRAP

Not more than 90 percent of the riprap stones shall be lighter than 150 pounds.

Not more than 50 percent of the riprap stones shall be lighter than 100 pounds.

Not more than 15 percent of the riprap stones shall be lighter than 40 pounds.

No riprap shall be lighter than 25 pounds.

TYPE II RIPRAP

All riprap stones shall be 150 pounds or less in weight except that occasional larger stones will be accepted provided they are no larger than the thickness of the riprap layer.

Not more than 50 percent of the riprap stones may be lighter than 60 pounds.

Not more than 15 percent of the riprap stones may be lighter than 25 pounds.

No riprap shall be lighter than 10 pounds.

ROCKFILL

Rockfill shall be a uniformly graded mixture of stones ranging in size from a minimum of 1 inch to a maximum of 12 inches.

Rock protection gradations.

Figure 2

61. With type I riprap around the four piers (plate 96), scour and rock movement following the SPF are shown on plate 97. Except for downstream pier 14, much of the bed material surrounding the riprap at all other piers was scoured away and the riprap was exposed; rock movement was minimal. Maximum scour of the bed adjacent to the riprap was to elevation 497.6 at upstream pier 14, to elevation 507.0 at downstream pier 14 (no exposed riprap), to elevation 495.3 at upstream pier 13, and to elevation 496.5 at downstream pier 13.

62. Timber debris (photograph 51) was allowed to collect on the four piers during the SPF, and the resulting scour is shown on plate 98. Maximum scour occurred to elevation 493.5 at the downstream pier 13--1.5 feet deeper than without debris. Slightly larger portions of the tops of the pier footings were exposed, and the extent of local scour was greater with the timber debris.

PART VI: PROJECT MODIFICATIONS TESTED

63. In the summer of 1981 an accumulation of silt, sand, and gravel became firmly compacted at the base of the outlet structure gate slots resulting in gate closure problems. A 1:50-scale model of a flat sill and gate slot similar to those of the Moose Creek outlet was used to study the problem. The material used in the model simulated the coarser material of the prototype but did not simulate the consolidation which would occur with the prototype material over a long period of time. However, the model did show that the material would be deposited in the slot and remain there when flow velocity was low--a condition that does occur at Moose Creek. A wedge sloped downward toward the sill at 30 degrees from the backside to the open edge of the slot, caused material to accumulate at the base of the wedge during low-velocity flow. The material accumulated at the edge of the main flow stream and was quickly swept away by the high-velocity flow created by either high discharge or insertion of a gate into the flow. The test indicated that such wedges could be effective in preventing the accumulation of fine material in the slots.

64. Subsequent to completion of model studies of the final-design diversion channel, the project design was modified to include a road embankment adjacent to the left overbank of the diversion channel from the existing Moose Creek dike to the Tanana River diversion dike (plate 99) to provide access to the diversion dike and overflow sill. Studies were conducted in the comprehensive model to determine what affect the access road would have on flow conditions in Pile Driver Slough, 15-Mile Slough, and in the diversion channel. Also studied were the maximum capacity flow conditions in the sloughs and the velocities along the face of Moose Creek Dam during the PMF.

65. Flow conditions with and without the access road were observed with the following flow distributions:

a. 100-year flood (40,500 cfs) in the diversion channel and coincident Tanana River discharge of 104,000 cfs.

b. PMF (160,000 cfs) in the diversion channel and coincident Tanana River discharges of 104,000 and 250,000 cfs.

c. Tanana River SPF (200,000 cfs) plus 40,000 cfs in Pile Driver Slough and either no flow or the 100-year flood in the diversion channel.

d. Tanana River SPF and the 100-year flood in the diversion channel (no inflow from Pile Driver Slough).

Water surface elevations both with and without the access road embankment for various flow conditions are summarized in table I. The maximum flow that would pass through Pile Driver and 15-Mile Sloughs without overtopping the access road was 2,500 cfs with a coincident Tanana River SPF discharge.

66. Velocities shown in table J were observed along the face of the Moose Creek Dam embankment at the locations shown on plate 99. The maximum velocity near the pipeline crossing (dam station 164+50) was slightly less than 5 fps. The highest velocities (up to 6.4 fps) occurred near the overflow sill at dam extension station 60+00.

PART VII: SUMMARY

67. Final hydraulic design of the Moose Creek Dam outlet works and diversion channel was accomplished using six separate fixed and movable-bed models. Modifications to the original-design outlet structure and channel were required to provide acceptable energy dissipation and to maintain acceptable water surface gradients through the modified outlet channel. The models were used to develop improved designs of the diversion channel bridge opening training dikes and the overflow sill into the Tanana River. Scour around the piers and abutments of bridges crossing the diversion channel was evaluated with movable-bed models.

TABLE A

SUMMARY OF TEST RESULTS
Stilling Basin Plans 1 to 14

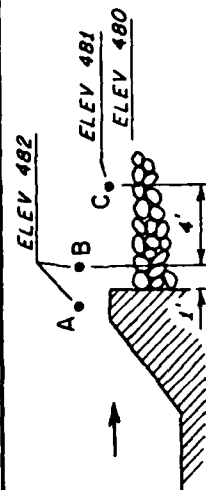
Outlet Discharge 9,000 CFS, Pool Elev 523.1, Normal Tailwater Elev 494.4

Plan No.	Dis-charge in CFS per Bay	Tail-water Elev in Feet M.S.L.	Velocity in FPS			Pressures on Baffle					Water-Surface Elev at Gate in Feet M.S.L.	Feet From Gate to Toe of Jump	Wave Height in Feet 150 Ft Down-stream of Gate
						Piezometer Number							
			Location**			B-1	B-2	B-3	B-4	B-5			
			A	B	C								
1	2250	494.4	17		10-12						484.0		
2	2250	494.4			U 7- 8						487.0		3.0
	3000	494.4			U 7- 9						484.4		3.5
3	2250	494.4			U 7- 9						489.0		3.0
4	2250	494.4			5- 9						486.0		2.0
5	2250	494.4			5- 8						487.0		3.0
6	2250	494.4			U 3	-17	33	-27	-14	-18	489.5		1.0
	3000	492.4			5	-21	40	-27	-18	-27	485.0		2.0
7	3000	494.4	8-10		4						486.0		3.0
8	2250	494.4	4		3	4	21	-7	-1	-2	489.0		1.5
	2250	492.4	4- 6		4	2	20	-6	-2	-3	484.0		2.0
	3000	494.4	9		4- 5	-2	25	-16	-8	-11	486.0		2.0

7	3000	494.4	8-10		4							486.0		3.0
8	2250	494.4	4		3	4	21	-7	-1	-2		489.0		1.5
	2250	492.4	4-6		4	2	20	-6	-2	-3		484.0		2.0
	3000	494.4	9		4-5	-2	25	-16	-8	-11		486.0		2.0
	3000	492.4	10-12		5-7	-16	36	-26	-22	-26	22			2.5
9	2250	494.4						0		-2		490.0		
	2250	492.4						-1		-2		485.0		
	3000	494.4						-6		-8		485.5		
	3000	492.4						-15		-21	13			
10	2250	494.4	6-7	5-7	4	2		2		0		489.0		
	2250	492.4	8-9	5-6	4	4		0		-2		485.0		
	3000	494.4	7-8	4-6	4	4		-4		-5		486.0		
	3000	492.4	10-11	5-7	3-5	1		-16		-18	15			
11	3000	492.4	7		3-4					-18				
12 13	3000	492.4						-14		-14				
14*	2250	494.4	5-8	4-6	3-4	-1	19	-4	-4	-2		490.0		1.0
	2250	492.4	7-8	5-6	4-5	-2	18	-4	-4	-1		486.5		1.5
	3000	494.4	7-8	7-8	3-5	-7	22	-11	-10	-7		488.0		2.0
	3000	492.4	10	7-8	4-5	-11	22	-14	-14	-10	5			1.5

* Recommended design

NOTES: 1. Piezometer locations shown on plate 5.
2. Velocities are downstream unless noted otherwise.



**VELOCITY OBSERVATIONS

TABLE A

TABLE B

SUMMARY OF TEST RESULTS
Stilling Basin Plans 14, 15, and 16
Single-bay Model

Outlet Discharge 9,000 CFS, Pool Elev 523.1, Normal Tailwater Elev 493.4

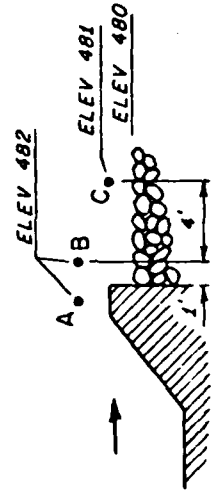
Plan No.	Dis-charge in CFS per Bay	Tail-water Elev in Feet M.S.L.	Velocity in FPS			Pressures on Baffle					Water-Surface Elev at Gate in Feet M.S.L.	Feet From Gate to Toe of Jump			
						Piezometer Number									
			Location**			B-1	B-2	B-3	B-4	B-5			Head on Piez. in Feet of Water		
			A	B	C										
14*	2250	494.4	5- 8	4- 6	3- 4	-1	19	-4	-4	-2	490.0	5			
	2250	492.4	7- 8	5- 6	4- 5	-2	18	-4	-4	-1	486.5				
	3000	494.4	7- 8	7- 8	3- 5	-7	22	-11	-10	-7	488.0				
	3000	492.4	10	7- 8	4- 5	-11	22	-14	-14	-10					
15	2250	494.4	3- 5	3- 5	3- 4	6	19	1	3	4	490.0	5			
	2250	493.4	5- 6	4- 5	4	4	18	0	2	2	489.0				
	2250	491.4	5- 8	5- 6	4- 5	2	18	-4	-2	0					
	3000	494.4	4- 6	4- 5	3- 4	2	22	-4	-1	-1	485.0				
	3000	493.4	5- 7	5- 7	3- 5	-2	23	-8	-6	-4					
	3000	491.4	10-12	8-10	7- 8	-14	29	-24	-20	-16					
16**	2250	494.4	5- 6	4- 5	4- 5	6	19	2	3	4	490.0	6			
	2250	493.4	5- 6	5- 6	4- 5	5	19	1	2	3	487.0				
	2250	491.4	6- 8	6- 7	5- 7	1	18	-3	-1	1					
	3000	494.4	6- 9	6- 8	4- 7	1	22	-5	-3	-1	484.0				
	3000	493.4	7- 9	6- 8	4- 5	-2	22	-6	-5	-2					
	3000	491.4	10-13	8-11	8-11	-16	23	-22	-20	-11					

* Recommended design

** Revised final design

NOTES: 1. Piezometer locations shown on plate 5.

2. Velocities are downstream.



**VELOCITY OBSERVATIONS

TABLE C

OUTLET GATE DISCHARGE RATING

Outlet Discharge CFS	Discharge per Bay CFS	Gate Opening ft	Pool Elev ft, MSL	Tailwater Elev ft, MSL	Water-Sur- face Elev Downstream Gate Face ft, MSL	Coef- ficient of Discharge
2,600	650	0.90	507.22	487.60	486.1	0.812
3,000	750	0.90	514.50	488.15	486.5	0.809
3,500	875	0.90	524.75	488.85	486.6	0.809
5,030	1,258	1.85	506.92	490.40	488.7	0.833
6,080	1,520	1.85	516.22	491.30	489.2	0.820
6,500	1,625	1.85	520.28	491.65	489.3	0.818
6,500	1,625	2.33	509.00	491.65	489.3	0.833
7,200	1,800	2.33	513.80	492.15	489.6	0.827
8,000	2,000	2.33	519.58	492.65	489.6	0.824
7,200	1,800	2.60	509.10	492.15	489.7	0.837
8,000	2,000	2.60	513.64	492.65	489.7	0.836
9,000	2,250	2.60	519.42	493.24	489.3	0.842
9,500	2,375	2.60	522.46	493.50	489.4	0.845

NOTES:

1. Data obtained from 1:20-scale single-bay model.
2. Tailwater elevations from 1:40-scale model at downstream end of outlet structure (Plan B outlet channel).
3. Data shown on plate 11.

Discharge coefficients: $Q = CA \sqrt{2g(d_u + h_v - d_d)}$

where: Q = Discharge per bay, in cfs
 C = Coefficient of discharge
 A = Area of gate opening, ft²
 g = Acceleration of gravity, ft/sec²
 d_u = Depth above invert upstream, ft
 h_v = Velocity head upstream of gate, ft
 D_d = Depth above invert, downstream from jump, ft

TABLE C

TABLE D

WATER-SURFACE ELEVATIONS
Chena River and Plan B Outlet Channel

Station	Gage No.	Water Surface							
		Base Elev* ft, MSL	Model Elev ft, MSL	Differ- ence ft.	Base Elev* ft, MSL	Model Elev ft, MSL	Differ- ence ft		
		Discharge 1,120 CFS			Discharge 4,000 CFS			Discharge 8,000 CFS	
R130+25	1	488.4	488.1	-0.3	--	492.4	--	496.4	--
C 42+10	F-1	486.6	486.8	+0.2	--	491.6	--	495.5	--
C 37+25	F-2	486.3	486.6	+0.3	--	491.2	--	494.8	--
C 33+25	F-3	486.3	486.4	+0.1	--	491.0	--	494.7	--
C 29+50	F-4	486.2	486.4	+0.2	--	490.8	--	494.5	--
C 25+55	F-5	486.2	486.3	+0.1	--	490.5	--	494.3	--
C 21+30	F-6	485.7	485.8	+0.1	--	490.0	--	494.0	--
C 17+30	F-7	485.5	485.6	+0.1	--	489.6	--	493.7	--
C 13+09	F-8	485.2	485.4	+0.2	--	489.2	--	493.2	--
Outlet	-	485.2	485.2	0	--	489.2	--	493.0	--
C 6+88	T-1	485.0	485.2	+0.2	--	489.0	--	492.4	--
C 4+12	T-2	484.9	485.0	+0.1	--	488.7	--	492.3	--
C 1+50	T-3	484.8	484.8	0	--	488.4	--	492.1	--
R 72+25	5	484.8	484.8	0	488.5	488.4	-0.1	492.1	--
R 63+00	6	484.2	484.3	+0.1	488.1	488.1	0	--	--
R 53+00	7	483.8	484.0	+0.2	487.7	487.6	-0.1	--	--
R 44+00	8	483.6	483.6	0	487.1	487.2	+0.1	--	--
		Discharge 1,630 CFS			Discharge 6,080 CFS			Discharge 9,000 CFS	
R130+25	1	489.3	489.0	-0.3	494.5	494.7	+0.2	497.4	-0.3
C 42+10	F-1	487.8	487.9	+0.1	493.8	493.8	0	496.0	+0.3
C 37+25	F-2	487.5	487.7	+0.2	493.2	493.4	+0.2	495.5	+0.3
C 33+25	F-3	487.4	487.5	+0.1	493.1	493.2	+0.1	495.3	+0.3
C 29+50	F-4	487.3	487.4	+0.1	492.9	493.0	+0.1	495.2	+0.1
C 25+55	F-5	487.2	487.2	0	492.7	492.8	+0.1	494.9	+0.2
C 21+30	F-6	486.8	486.8	0	492.4	492.3	-0.1	494.6	-0.2
C 17+30	F-7	486.5	486.4	-0.1	492.0	492.0	0	494.2	0
C 13+09	F-8	486.2	486.1	-0.1	491.6	491.5	-0.1	493.7	0
Outlet	-	486.1	486.0	-0.1	491.5	491.3	-0.2	493.5	0
		Discharge 8,000 CFS			Discharge 9,000 CFS			Discharge 10,000 CFS	
R130+25	1	489.3	489.0	-0.3	497.4	497.1	-0.3	499.4	-0.3
C 42+10	F-1	487.8	487.9	+0.1	496.0	496.3	+0.3	498.0	+0.3
C 37+25	F-2	487.5	487.7	+0.2	495.5	495.8	+0.3	497.5	+0.3
C 33+25	F-3	487.4	487.5	+0.1	495.3	495.6	+0.3	497.3	+0.3
C 29+50	F-4	487.3	487.4	+0.1	495.2	495.3	+0.1	497.2	+0.1
C 25+55	F-5	487.2	487.2	0	494.9	495.1	+0.2	497.1	+0.2
C 21+30	F-6	486.8	486.8	0	494.6	494.4	-0.2	496.8	-0.2
C 17+30	F-7	486.5	486.4	-0.1	494.2	494.2	0	496.5	0
C 13+09	F-8	486.2	486.1	-0.1	493.7	493.7	0	496.2	0
Outlet	-	486.1	486.0	-0.1	493.5	493.5	0	496.0	0

	Discharge 1,630 CFS		Discharge 6,080 CFS		Discharge 9,000 CFS	
R130+25	489.3	489.0	494.5	494.7	497.4	497.1
C 42+10	487.8	487.9	493.8	493.8	496.0	496.3
C 37+25	487.5	487.7	493.2	493.4	495.5	495.8
C 33+25	487.4	487.5	493.1	493.2	495.3	495.6
C 29+50	487.3	487.4	492.9	493.0	495.2	495.3
C 25+55	487.2	487.2	492.7	492.8	494.9	495.1
C 21+30	486.8	486.8	492.4	492.3	494.6	494.4
C 17+30	486.5	486.4	492.0	492.0	494.2	494.2
C 13+09	486.2	486.1	491.6	491.5	493.7	493.7
Outlet	486.1	486.0	491.5	491.3	493.5	493.5
C 6+88	486.0	485.9	491.0	491.3	493.1	493.4
C 4+12	485.8	485.6	490.8	491.0	492.8	493.0
C 1+50	485.6	485.4	490.7	490.8	492.7	492.8
R 72+25	485.5	485.4	490.6	490.7	492.7	492.7
R 63+00	485.0	485.0	490.1	490.3	492.2	492.2
R 53+00	484.6	484.6	489.8	489.8	491.5	491.5
R 44+00	484.3	484.3	489.4	489.4	490.9	490.9
Discharge 2,660 CFS						
R130+25	490.7	490.8	--	495.4	--	499.0
C 42+10	489.8	489.8	--	494.7	--	498.0
C 37+25	489.4	489.5	--	494.2	--	497.5
C 33+25	489.2	489.2	--	494.0	--	497.3
C 29+50	489.1	489.2	--	493.8	--	497.0
C 25+55	488.9	489.0	--	493.5	--	496.6
C 21+30	488.5	488.5	--	493.0	--	496.0
C 17+30	488.1	488.2	--	492.7	--	495.6
C 13+09	487.7	487.8	--	492.2	--	495.0
Outlet	487.7	487.6	--	492.1	--	494.8
C 6+88	487.4	487.6	--	491.9	--	494.6
C 4+12	487.2	487.3	--	491.7	--	494.3
C 1+50	487.0	487.0	--	491.5	--	494.0
R 72+25	486.9	486.9	491.6	491.4	-0.2	493.9
R 63+00	486.5	486.4	490.9	491.2	+0.3	493.4
R 53+00	486.0	485.9	--	490.6	--	492.6
R 44+00	485.4	485.4	490.2	490.2	0	491.9

Notes:

1. Water surface profiles shown on plate 20.
2. River sta 0+00 at U.S.G.S. gage.
3. Model elevations not affected by estimated downstream channel realignment.

Legend:

- R River station
C Outlet channel station

* Gage 1 - Model verification data
"P" and "T" gages - Computed elevations
Gages 5 to 8 - Prototype data

TABLE D

TABLE E
FLOW DISTRIBUTION THROUGH OUTLET BAYS

Discharge in CFS			Distribution in Percent			
Total	¹ Fishways or Fish Ladder	Gate Bays ²	Bay 1	Bay 2	Bay 3	Bay 4
Ungated flow						
2,000	110	1,890	19	25	29	27
4,000	195	3,805	18	25	29	28
6,000	320	5,680	19	25	28	28
Gated Flow - Ungated Distribution						
2,000	83	1,920	19	25	29	27
4,000	114	3,885	18	25	29	28
6,000	138	5,860	19	25	28	28
Gated Flow - Linear Distribution						
2,000	83	1,920	21	24	26	29
4,000	114	3,885	21	24	26	29
6,000	138	5,860	20	23	27	30
Gated Flow - Uniform Distribution						
2,000	83	1,920	25	25	25	25
4,000	114	3,885	25	25	25	25
6,000	138	5,860	25	25	25	25

¹ Ungated flows only
² To nearest 5 cfs

TABLE E

TABLE F

TABLE F
OVERFLOW SILL SOUR

Flood Frequency	Undistorted Basin 1:12 Scale				Distorted Basin 1:100H, 1:25V Scale			
	At End Sill ft		Maximum ft		At End Sill ft		Maximum ft	
	Elev	Below Lip	Elev	Below Elev 502.0	Elev	Below Lip	Elev	Below Elev 502.0
10-year	500.0	0	499.8	2.2	500.0	0	499.8	2.2
100-year	500.0	0	498.5	3.5	500.0	0	499.0	3.0
SFF	500.0	0	497.5	4.5	500.0	0	497.5	4.5
PRF	500.0	0	493.0	9.0	499.5	0.5	493.0	9.0
End of Hydrograph	498.5	1.5	493.0	9.0	499.5	0.5	493.0	9.0

TABLE G
RAILROAD BRIDGE PIER TEST DATA

Flood Frequency	Discharge cfs/ft	Depth of Flow ft	Velocity fps	Elev of Maximum Local Scour ft	Depth of Maximum Local Scour ft	Remarks
100-yr	40	9.28	4.32	498.5	3.5	
SPF	80	11.92	6.70	496.5	5.5	
PMF	139	14.91	9.30	495.0*	7.0	
	80	10.00	8.00	496.8	5.2	
	93	10.00	9.30	495.0*	7.0	
	100	10.00	10.00	495.0*	7.0	
	120	14.90	8.00	495.0*	7.0	
	139	14.60	9.50	495.0*	7.0	
	144	15.50	9.30	495.0*	7.0	
PMF	139	14.91	9.30	495.0*	7.0	Timber debris on pier nose
	100	9.16	10.92	494.8*	7.2	Partial blockage of channel

* A portion of the top of the pier footing was exposed.

NOTES: 1. Flow and scour depths referenced to original bed elev 502.0.

2. Flow depths for the 100-yr, standard project (SPF), and probable maximum (PMF) floods based on diversion channel roughness coefficient of Manning's "n" = 0.032.

TABLE G

TABLE H

TABLE H
HIGHWAY BRIDGE PIER TEST DATA

Flood Frequency	Discharge cfs/ft	Angle of Approach Flow degrees	Depth of Flow ft	Velocity fps	Elev of Maximum Local Scour ft		Maximum Depth of Local Scour ft	Remarks
					Upstream Pier	Downstream Pier		
100-yr SPF PMF	44	0	9.84	4.50	499.2	499.2	2.8	Run 1) Timber debris on Run 2) upstream pier nose Manning's "n" = 0.026 Manning's "n" = 0.026 Fine bed material Type I rock protection Clear-water scour with type I rock protection
	80	0	12.60	6.39	497.7	498.0	4.3	
	147	0	17.10	8.60	498.0	497.0	5.0	
100-yr SPF PMF	44	15	9.84	4.50	497.0	498.0	5.0	
	80	15	12.60	6.39	495.3	496.0	6.7	
	147	15	17.10	8.60	494.6*	495.0*	7.4	
SPF SPF PMF	80	15	12.60	6.39	489.5*	496.8	12.5	
	80	15	12.60	6.39	489.5*	498.5	12.5	
	147	15	17.10	8.60	489.2*	498.0	12.8	
SPF PMF	81	15	11.77	6.85	495.8	496.2	6.2	Manning's "n" = 0.026 Manning's "n" = 0.026 Fine bed material Type I rock protection Clear-water scour with type I rock protection
	147	15	16.00	9.21	495.0*	495.0*	7.0	
SPF	80	15	12.60	6.39	495.3	495.4	6.7	
SPF	80	15	12.60	6.39	497.0	497.3	5.0	
SPF	80	15	12.60	6.39	496.9	497.4	5.1	

* A portion of the top of the pier footing was exposed.

NOTE: 1. Flow and scour depths referenced to original bed elev 502.0.

2. Flow depths based on diversion channel roughness coefficient of Manning's "n" = 0.032 unless otherwise indicated.

TABLE I

WATER-SURFACE ELEVATIONS
Moose Creek Diversion Channel

Plan B, With and Without Access Road/Dike

Model Gage ¹	Discharges Tanana River 200,000 CFS Diversion Channel 40,500 CFS ² Pile Driver Slough 40,000 CFS			Discharges Tanana River 200,000 CFS Diversion Channel 40,500 CFS ² Pile Driver Slough no flow			Discharges Tanana River 200,000 CFS Diversion Channel no flow Pile Driver Slough 40,000 CFS		
	Water-surface Elev		Change Due to Dike	Water-surface Elev		Change Due to Dike	Water-surface Elev		Change Due to Dike
	With Dike	Without Dike		With Dike	Without Dike		With Dike	Without Dike	
T-1	512.2	511.6	+0.6	510.2	510.4	-0.2	512.3	510.8	+1.5
T-2	512.2	511.8	+0.4	510.3	510.5	-0.2	512.4	511.0	+1.4
1	513.6	513.8	-0.2	512.8	512.6	+0.2	509.2	510.4	-1.2
2	513.4	513.6	-0.2	512.5	512.4	+0.1	509.2	510.4	-1.2
3	513.0	513.2	-0.2	511.7	511.7	0	509.2	510.4	-1.2
4	512.7	513.0	-0.3	511.5	511.3	+0.2	509.2	510.4	-1.2
5	512.5	512.9	-0.4	511.4	511.2	+0.2	509.2	510.4	-1.2
6	512.4	512.7	-0.3	511.2	511.1	+0.1	509.1	510.3	-1.2
7	512.1	512.4	-0.3	510.9	510.8	+0.1	509.0	510.1	-1.1
8	511.7	512.0	-0.3	510.7	510.6	+0.1	508.9	509.9	-1.0
9	511.6	511.9	-0.3	510.6	510.5	+0.1	508.8	509.8	-1.0
10	511.4	511.7	-0.3	510.4	510.4	0	508.8	509.7	-0.9
B-1	513.6	513.8	-0.2	512.7	512.6	+0.1	509.2	510.4	-1.2
B-2	513.9	513.0	+0.9	510.6	511.2	-0.6	514.0	510.9	+3.1
B-3	514.0	513.0	+1.0	510.6	511.1	-0.5	514.1	511.2	+2.9
B-4	513.3	512.6	+0.7	510.7	511.0	-0.3	513.3	511.3	+2.0
B-5	513.9	513.0	+0.9	510.6	511.1	-0.5	514.0	511.4	+2.6

¹ Gage locations shown on plate 95.

² 100-year flood.

TABLE I

TABLE J

VELOCITIES ALONG FACE OF MOOSE CREEK DAM
Plan B, Rising and Stabilized Flow Conditions, Probable Maximum Flood

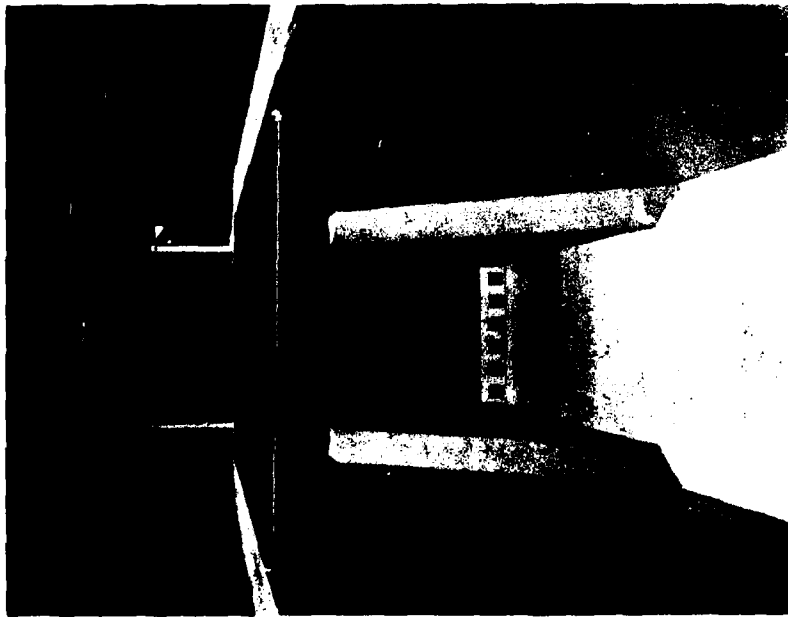
Floodway Dis- charge CFS	Velocity in FPS							
	Transient (rising flow)		Maximum Flow (stabilized)		Transient (rising flow)		Maximum Flow (stabilized)	
	T*	B*	T	B	T	B	T	B
	Station 168+00				Station 164+50**			
40,000	1.0	---			1.8	---		
70,000	1.4	1.0			2.6	2.6		
100,000	1.4	1.4			3.0	3.0		
130,000	1.4	1.4			3.4	3.0		
160,000	1.8	1.8	1.8	1.4	4.7	4.3	4.3	3.8
	Station 157+00				Station 145+00			
43,000	1.0	---			1.4	---		
71,600	1.0	1.0			1.4	1.4		
102,900	1.0	1.0			1.8	1.4		
126,400	1.0	1.0			1.8	1.4		
160,000	1.0	1.0	1.0	1.0	1.8	1.4	1.6	1.4
	Station 130+00				Station 117+50			
45,600	<1.0	---			<1.0	---		
74,200	1.0	1.0			1.0	1.0		
105,500	1.0	1.0			1.4	1.0		
131,500	1.0	1.0			1.4	1.4		
160,000	1.0	1.0	1.0	1.0	1.8	1.4	1.8	1.4
	Ex Station 100+00							
45,600	Slack	Slack						
74,200	Slack	Slack						
105,500	1.4	<1.0						
131,500	2.6	<1.0						
160,000	3.8	3.8	4.3	3.8				
	Ex Station 80+00				Ex Station 60+00			
48,200	1.0	---			3.4	---		
76,900	1.8	1.4			4.3	3.8		
108,100	2.2	1.8			4.3	4.7		
134,100	2.6	2.2			6.0	6.0		
160,000	2.6	2.2	3.0	1.8	6.4	6.4	6.0	6.0

* T, 3-ft depth; B, 3 ft above toe of dam.

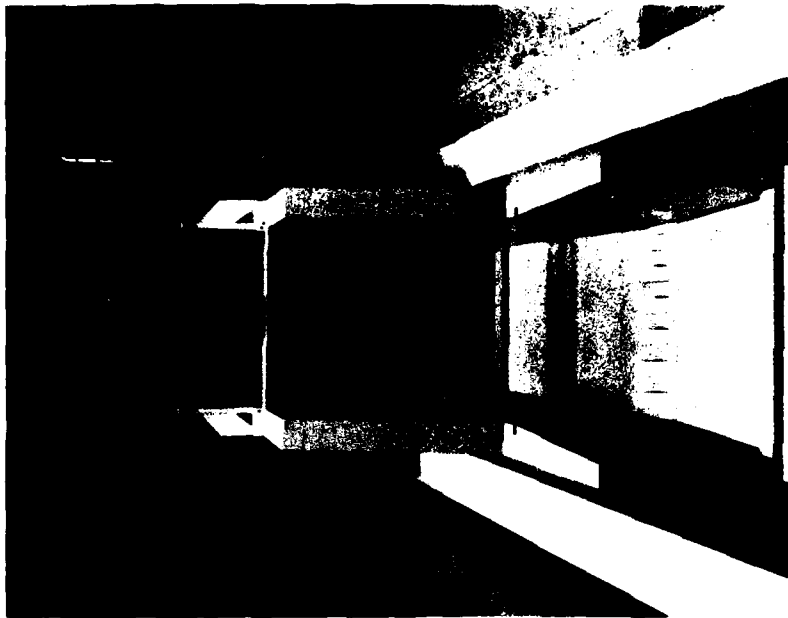
** Velocities measured at upstream outer corner of pipeline covering.

- NOTES: 1. Metering stations shown on plate 96.
 2. Rate of rise of transient flow, 130 cfs/min (from Chart 4, DM No. 10, Vol I, 28 May 74).
 3. Except for stations 130+00 and 117+00, all flow along the face of the dam was downstream

TABLE J

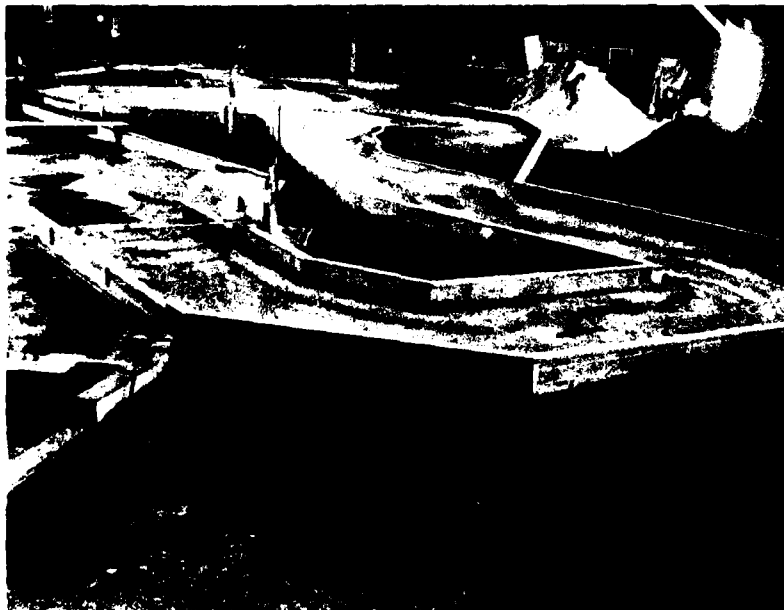


Looking downstream

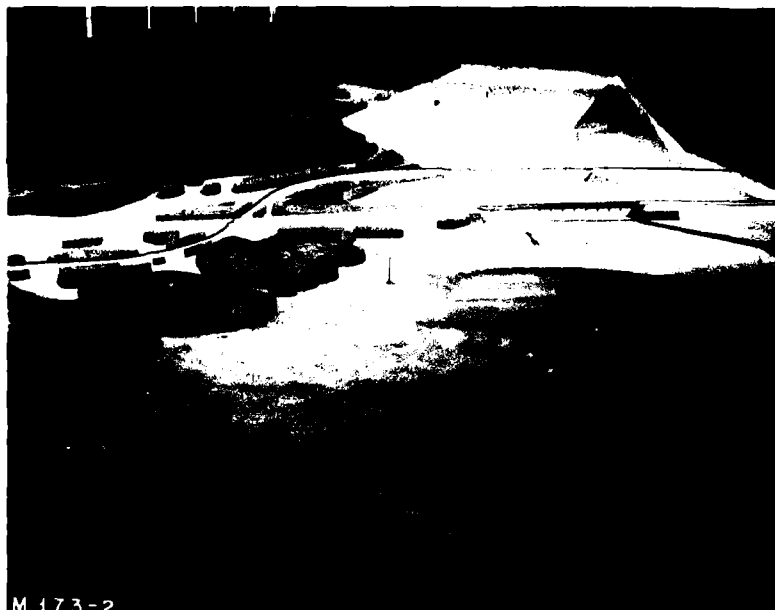


Looking upstream

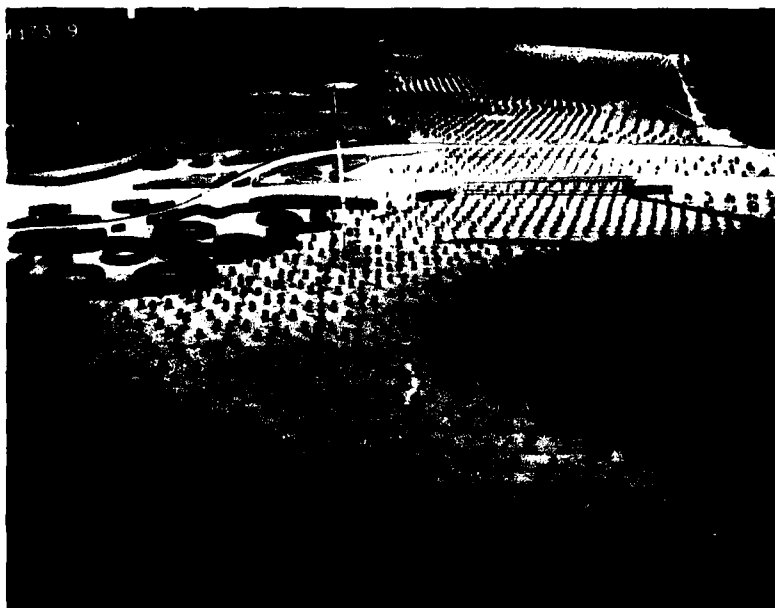
Photograph 1. Single bay outlet structure model with original design (Plan 1) stilling basin.



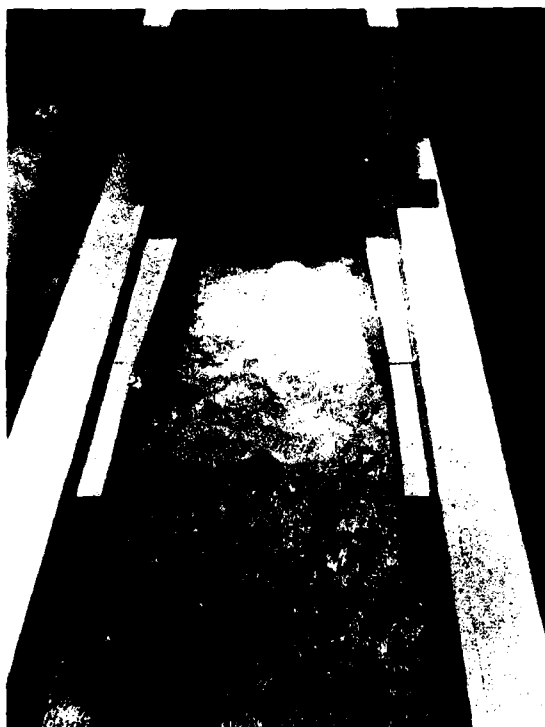
Photograph 2. General views of outlet channel model prior to installation of dam and outlet structure.



M 473-2
 Photograph 3. General view of diversion channel comprehensive model prior to installation of roughness elements.



Photograph 4. Diversion channel roughness elements.



Photograph 5. Flow conditions in plan 14 stilling basin with extreme operating condition. Outlet discharge 9,000 cfs (3,000 cfs per bay), pool elevation 523.1, tailwater elevation 492.4.



Photograph 6. Flow conditions in plan 15 stilling basin. Outlet discharge 9,000 cfs (3,000 cfs per bay--one non-operative bay). Pool elevation 523.1, tailwater elevation 491.4 (2 feet below normal).



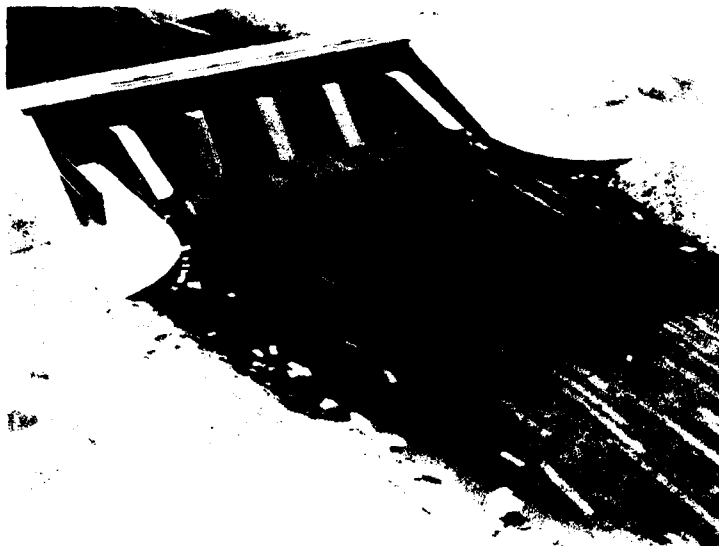
Photograph 7. Plan A (original design) outlet structure and fishway entrance.



Photograph 8. Plan B outlet structure with plan A fishway entrance.



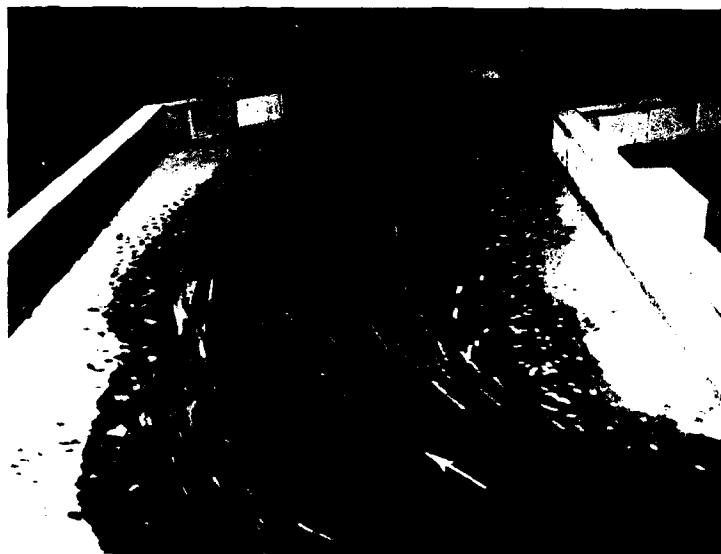
Photograph 9. Surface flow conditions in plan B outlet channel upstream from ungated outlet structure, river discharge 9,000 cfs.



Photograph 10. Surface flow conditions in plan B outlet channel approach to ungated plan C outlet structure, river discharge 9,000 cfs.

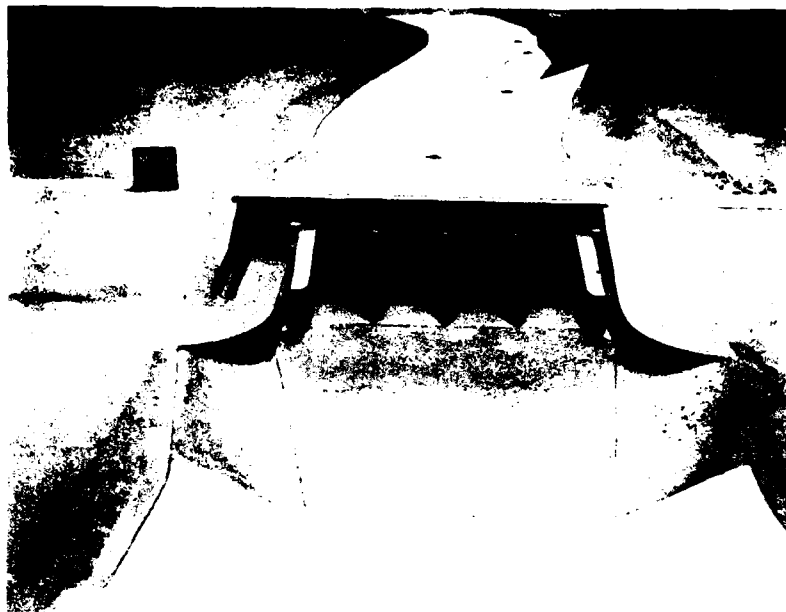


Photograph 11. Downstream view from station 57+85.



Photograph 12. Upstream view from station 57+85.

Surface flow conditions in downstream river channel portion of the plan B outlet channel with estimated channel realignment, river discharge 9,000 cfs.



Photograph 13. Plan C (final design) outlet structure with plan B fishway entrance.



Plan B fishway entrance



Plan C fishway entrance

Photograph 14. Flow conditions in left fishway entrance, plan B outlet channel, plan C outlet structure, ungated outlet discharge 2,660 cfs, tailwater elevation (gage T-1) 487.6.



Plan B fishway entrance



Plan C fishway entrance

Photograph 15. Flow conditions in left fishway entrance, plan B outlet channel, plan C outlet structure, ungated outlet discharge 4,000 cfs, tailwater elevation (gage T-1) 489.0.

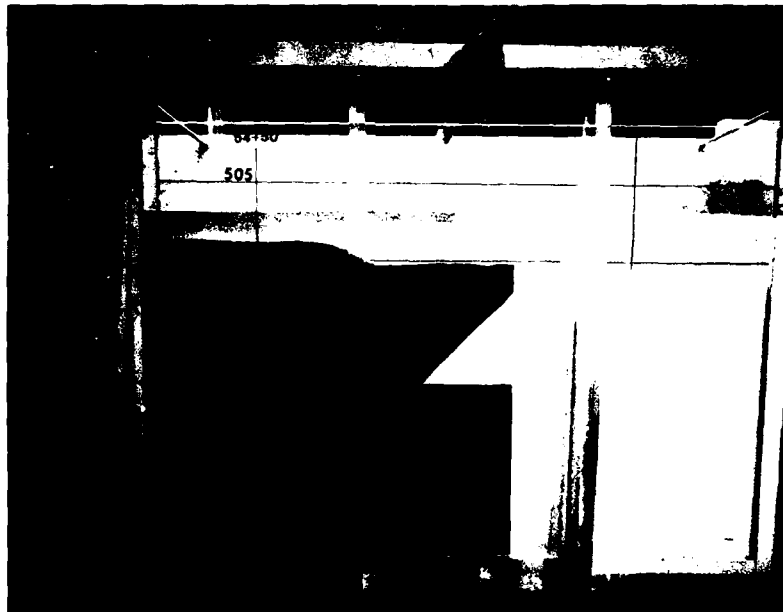


Plan B fishway entrance

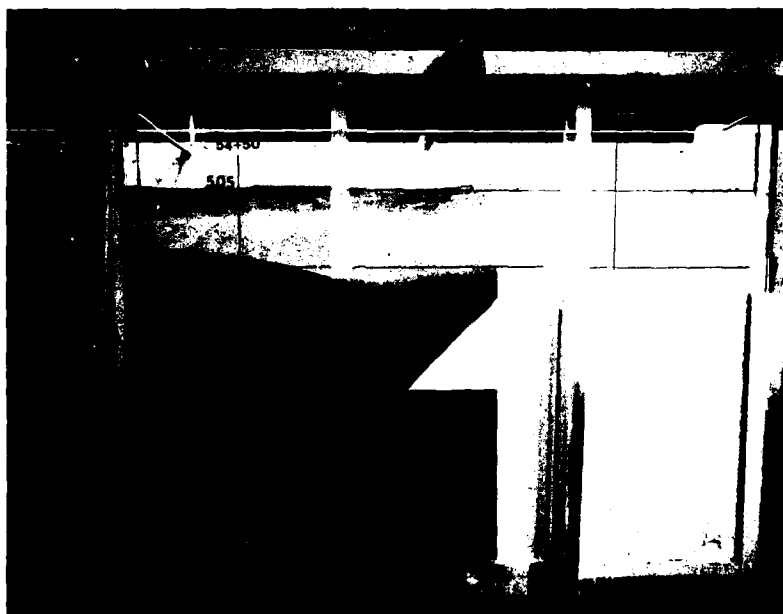


Plan C fishway entrance

Photograph 16. Flow conditions in left fishway entrance, plan B outlet channel, plan C outlet structure, ungated outlet discharge 6,080 cfs, tailwater elevation (gage T-1) 491.3.

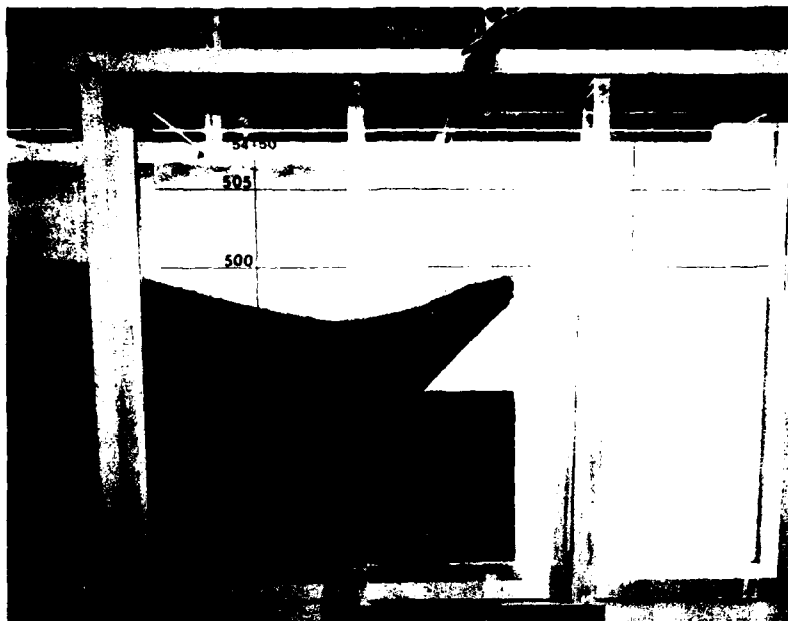


10-year flood, tailwater elevation 503.2.
Tanana River water-surface elevation 499.0.

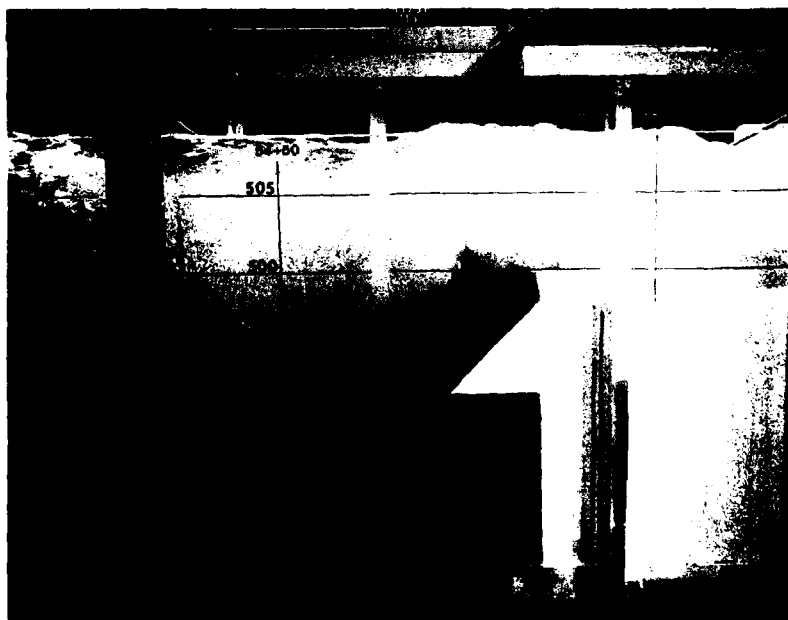


100-year flood, tailwater elevation 505.0.
Tanana River water-surface elevation 503.0.

Photograph 17. Sheet pile overflow sill with plan B basin,
undistorted 1:12-scale model. Flow conditions
and scour profile, PMF hydrograph.



Standard project flood, tailwater elevation 506.0.
Tanana River water-surface elevation 503.0.

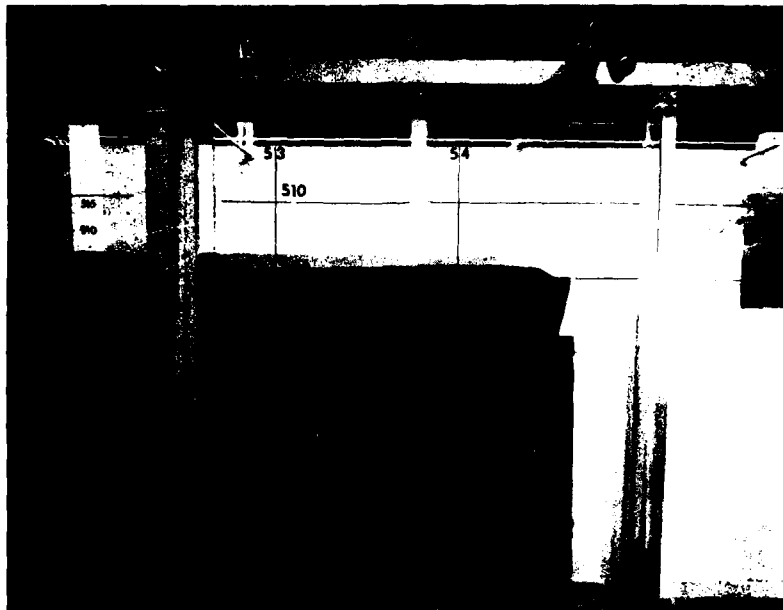


Probable maximum flood, tailwater elevation 508.7.
Tanana River water-surface elevation 503.0.

Photograph 18. Sheet pile overflow sill with plan B basin,
undistorted 1:12-scale model. Flow conditions
and scour profile, PMF hydrograph.



Photograph 19. Sheet pile overflow sill with plan B basin, undistorted 1:12-scale model. Scour profile at end of test, PMF hydrograph.

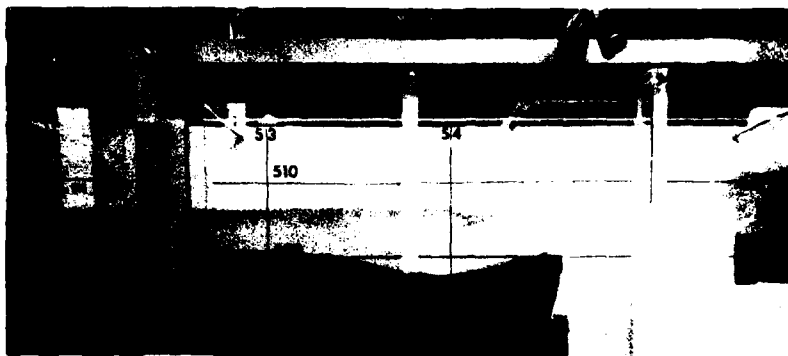


10-year flood, tailwater elevation 503.2.
Tanana River water-surface elevation 499.0.

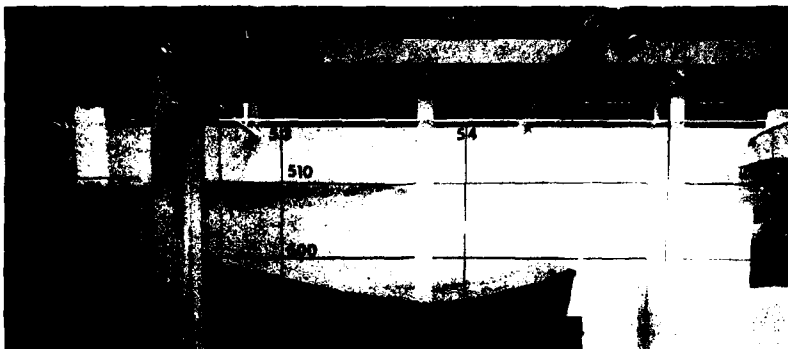


100-year flood, tailwater elevation 505.0.
Tanana River water-surface elevation 503.0.

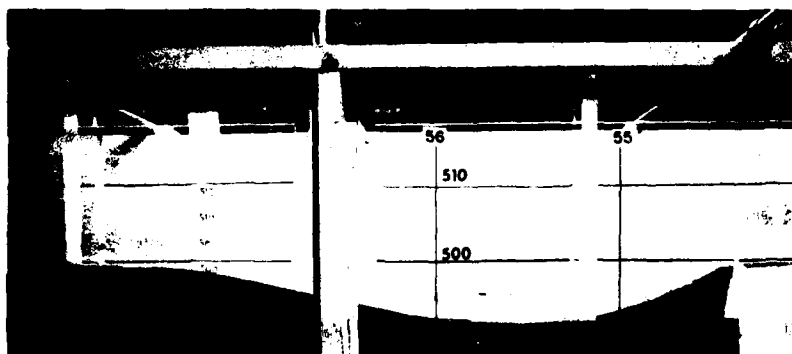
Photograph 20. Sheet pile overflow sill with plan B basin,
distorted scale 1:100H, 1:25V. Flow conditions
and scour profile, PMF hydrograph.



Standard project flood, tailwater elevation 506.0
Tanana River water surface elevation 503.0.

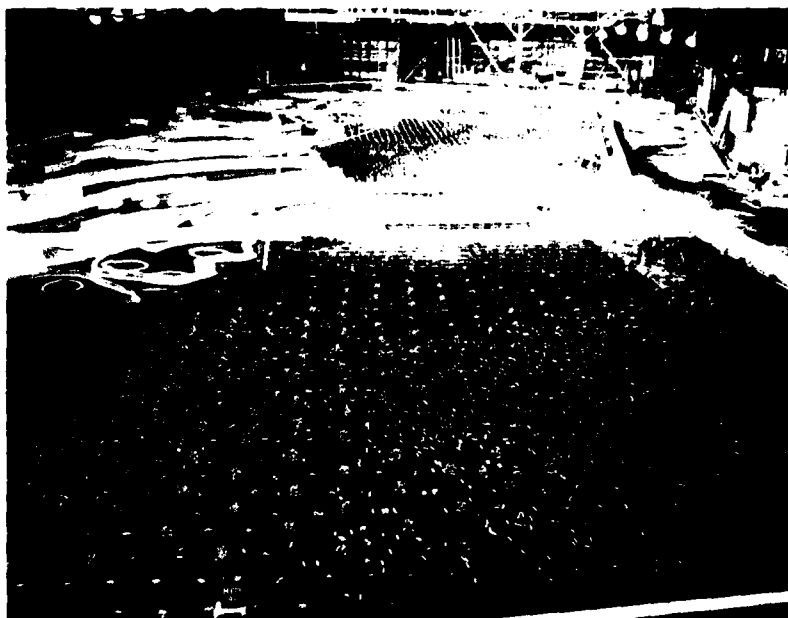


Probable maximum flood, tailwater elevation 508.7
Tanana River water surface elevation 503.0.

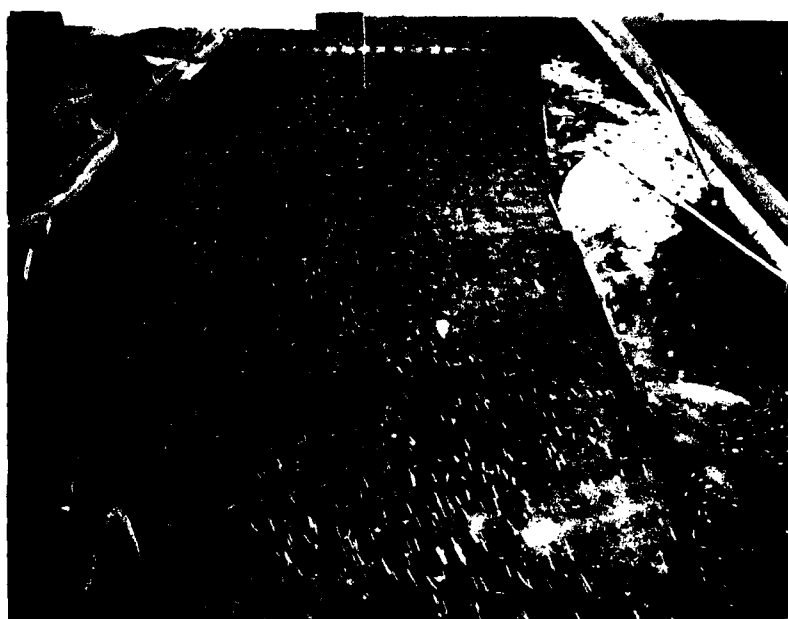


Scour profile at end of PMF hydrograph.

Photograph 21. Sheet pile overflow sill with plan B basin in distorted scale model (1:100H, 1:25V). Flow conditions and scour profile, PMF hydrograph.



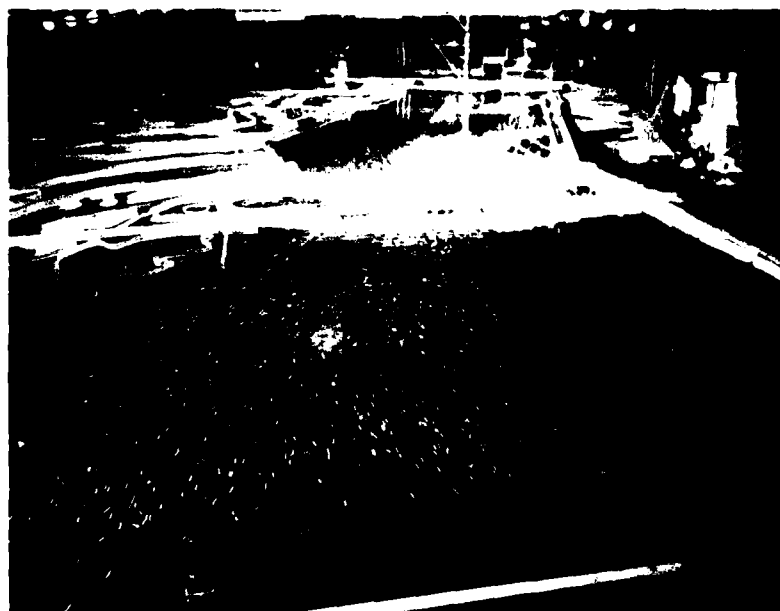
Upstream of bridges



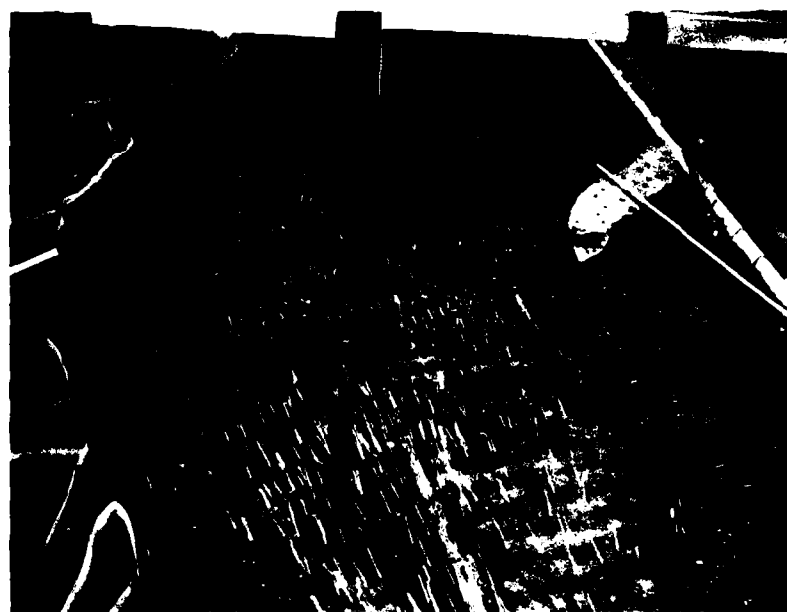
Downstream of bridges

Plan B

Photograph 22. Flow conditions in diversion channel.
10-year flood; floodway discharge 15,030 cfs.



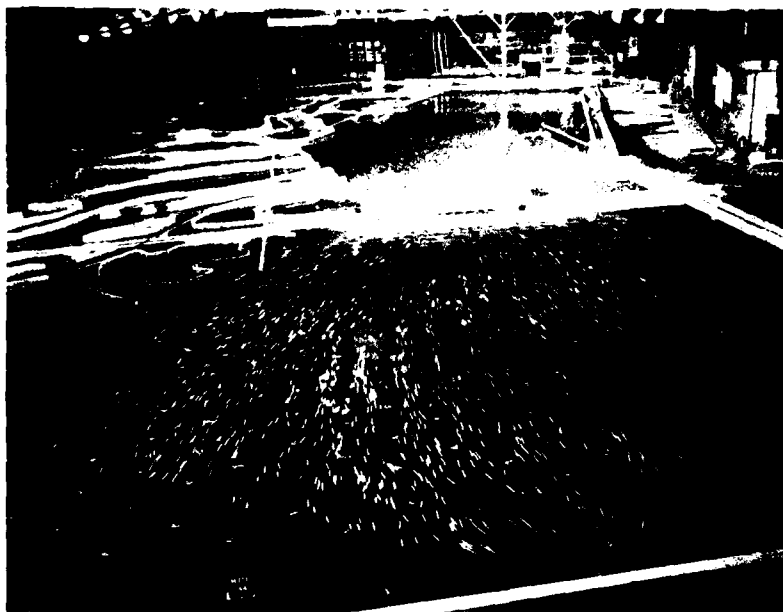
Upstream of bridges



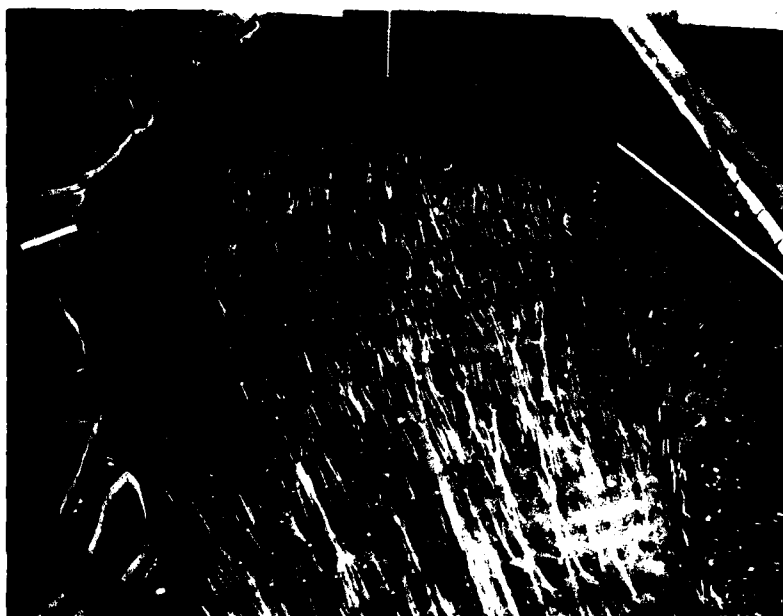
Downstream of bridges

Plan B

Photograph 23. Flow conditions in diversion channel.
100-year flood; floodway discharge 40,500 cfs.



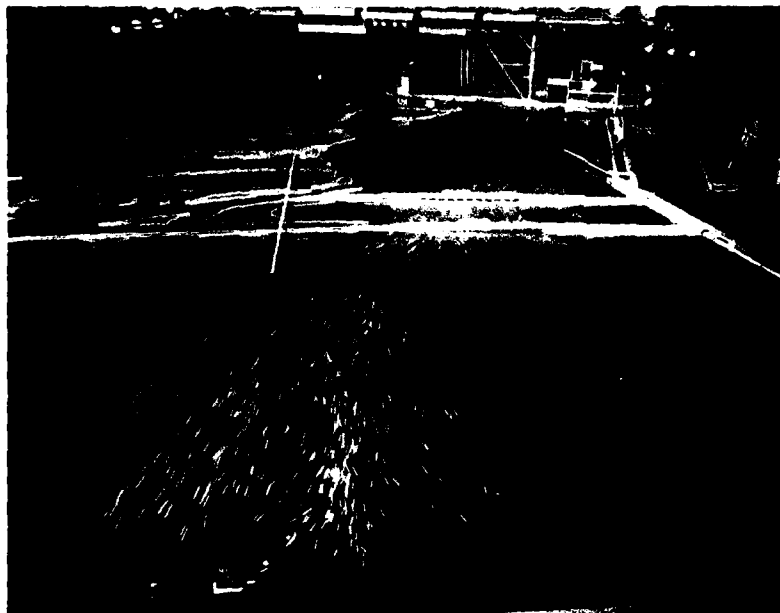
Upstream of bridges



Downstream of bridges

Plan B

Photograph 24. Flow conditions in diversion channel.
Standard project flood; floodway
discharge 74,000 cfs.



Upstream of bridges



Downstream of bridges

Plan B

Photograph 25. Flow conditions in diversion channel.
Probable maximum flood; floodway
discharge 160,000 cfs.



Standard project flood
Floodway discharge 74,000 cfs.



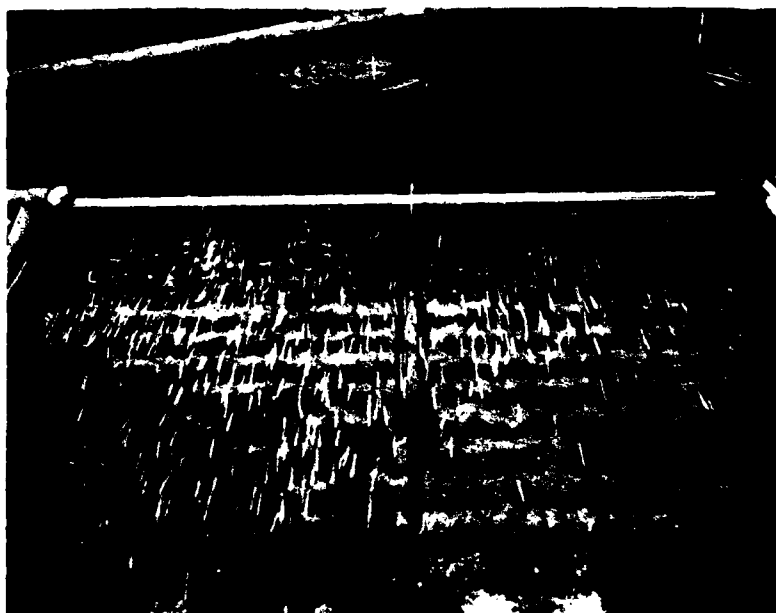
Probable maximum flood
Floodway discharge 160,000 cfs.

Plan B

Photograph 26. Flow over highway and railroad at crossing.



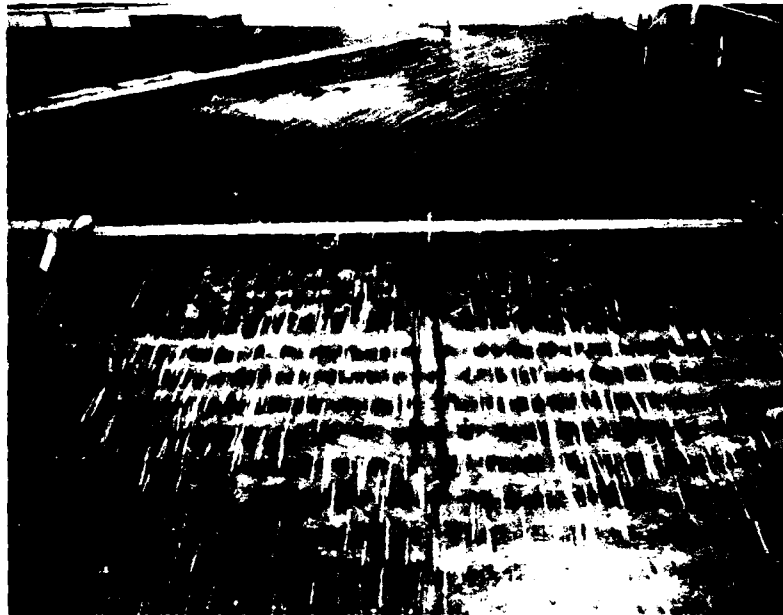
10-year flood
Floodway discharge 15,030 cfs
Tanana River discharge 75,000 cfs.



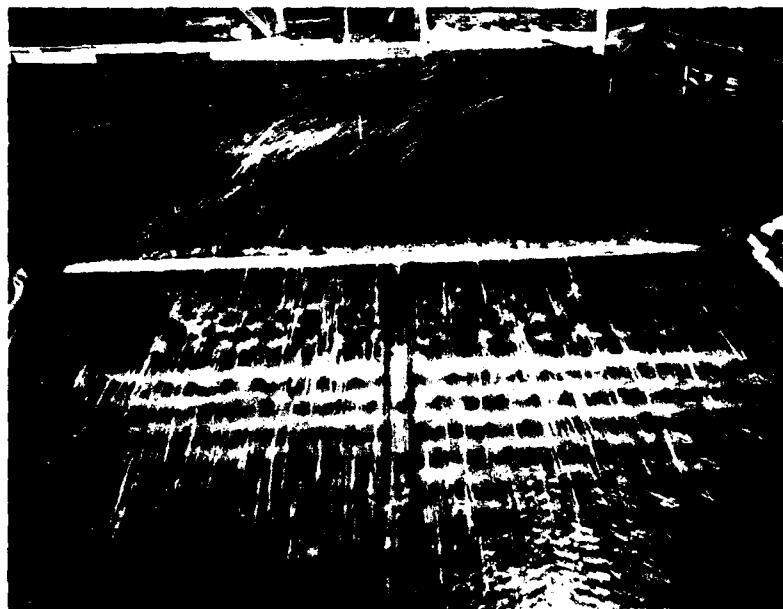
100-year flood
Floodway discharge 40,500 cfs
Tanana River discharge 104,000 cfs.

Plan B

Photograph 27. Flow conditions at overflow sill, looking downstream.



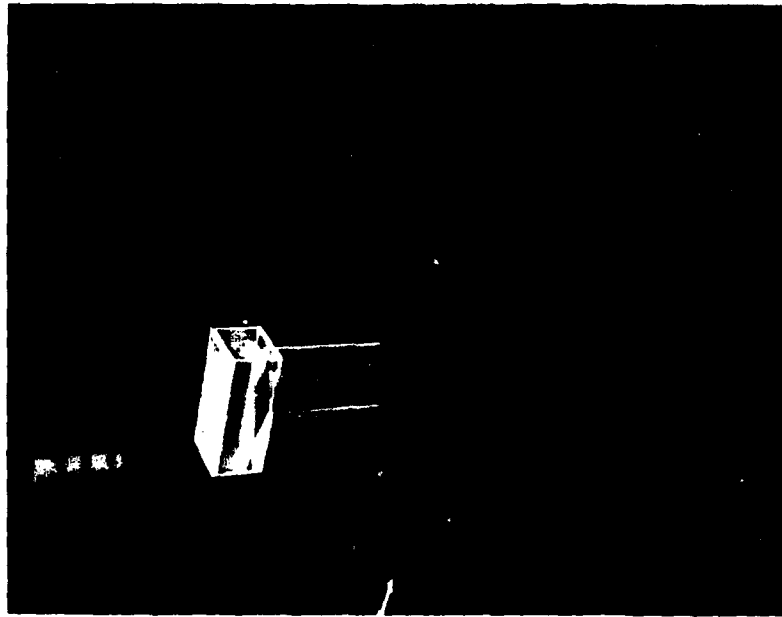
Standard project flood
Floodway discharge 74,000 cfs
Tanana River discharge 125,000 cfs.



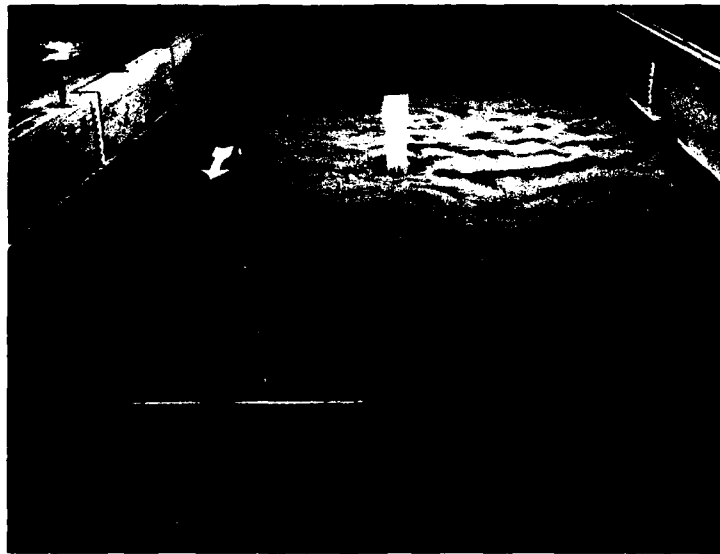
Probable maximum flood
Floodway discharge 160,000 cfs
Tanana River discharge 250,000 cfs.

Plan B

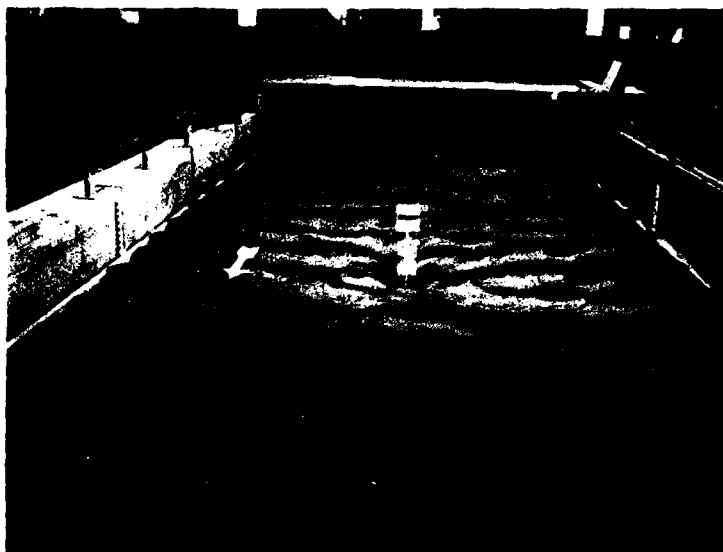
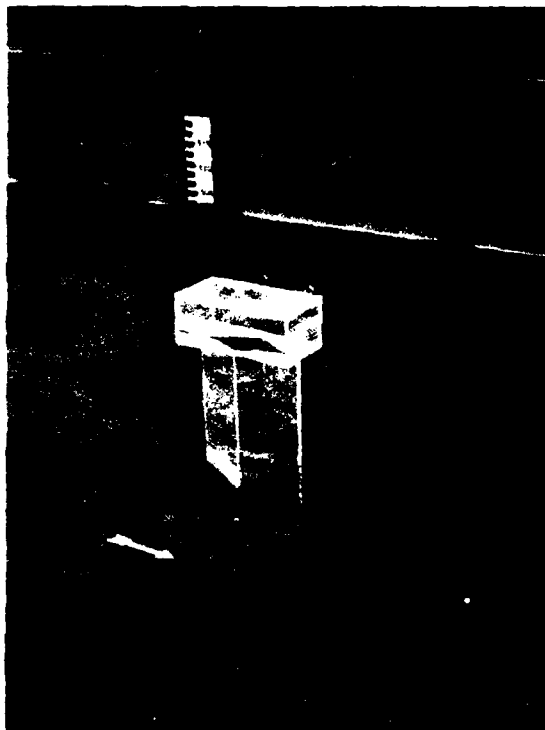
Photograph 28. Flow conditions at overflow sill, looking downstream.



Photograph 29. Scour patterns, railroad bridge pier. 100-year flood, discharge 40 cfs/foot, velocity 4.32 fps, depth 9.28 feet. Duration 3 hrs 52 min.



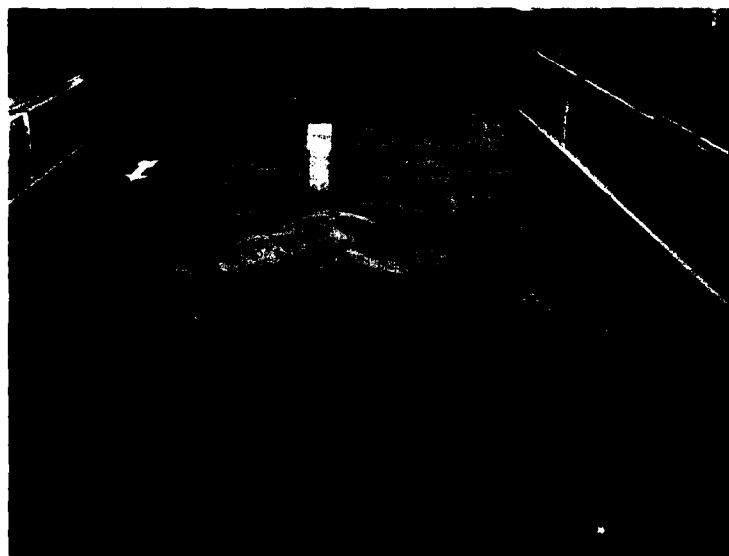
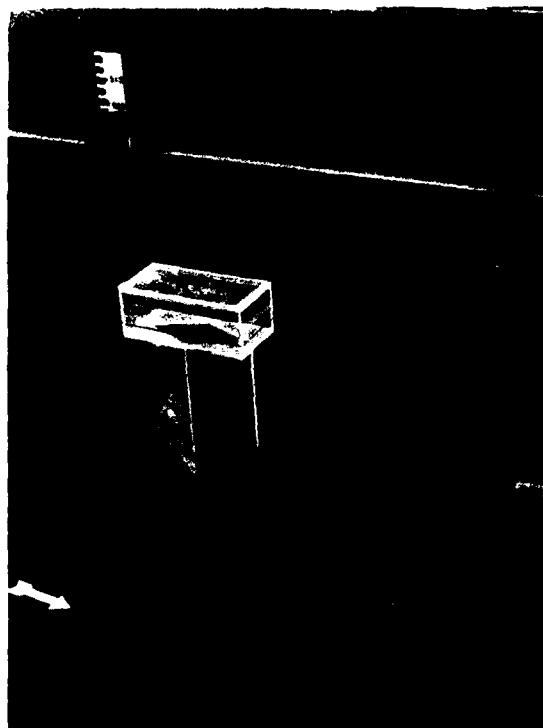
Photograph 30. Scour patterns, railroad bridge pier.
Standard project flood, discharge 80 cfs/foot,
velocity 6.70 fps, depth 11.92 feet. Duration
1 hr 56 min.



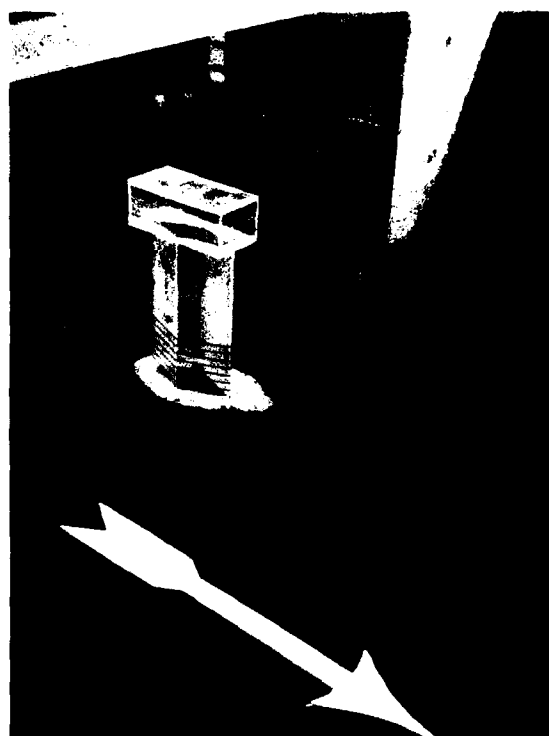
Photograph 31. Scour patterns, railroad bridge pier. Probable maximum flood, discharge 139 cfs/foot, velocity 9.30 fps, depth 14.91 feet. Duration 1 hr 56 min.



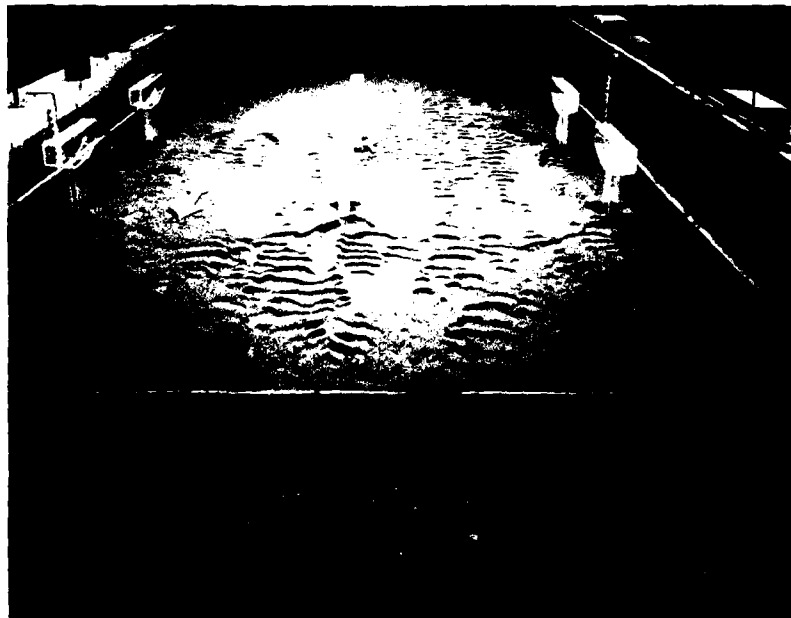
Photograph 32. Timber debris collected on railroad bridge pier. Probable maximum flood.



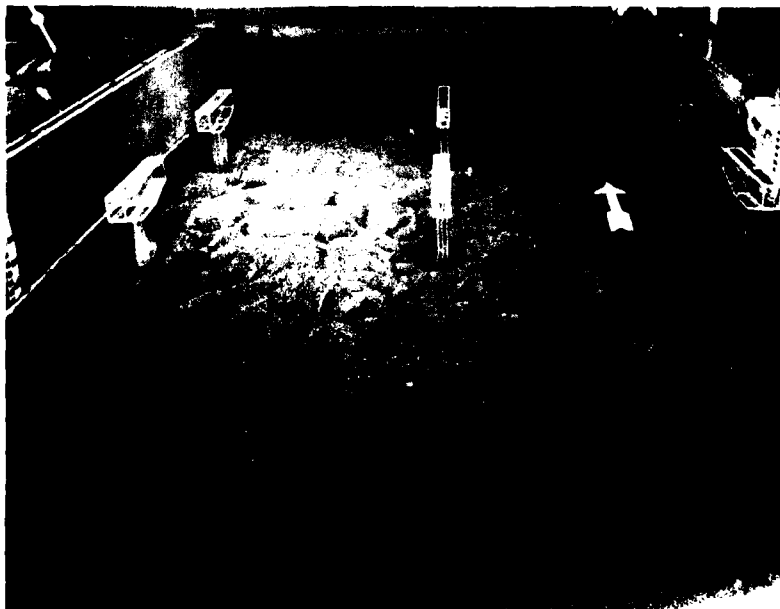
Photograph 33. Scour patterns, railroad bridge pier with timber debris. Probable maximum flood, discharge 139 cfs/foot, velocity 9.30 fps, depth 14.91 feet. Duration 1 hr 56 min.



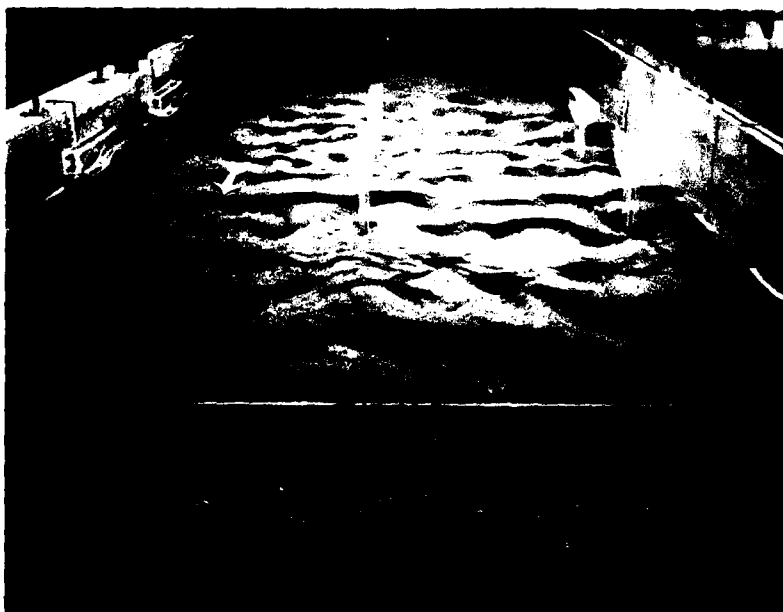
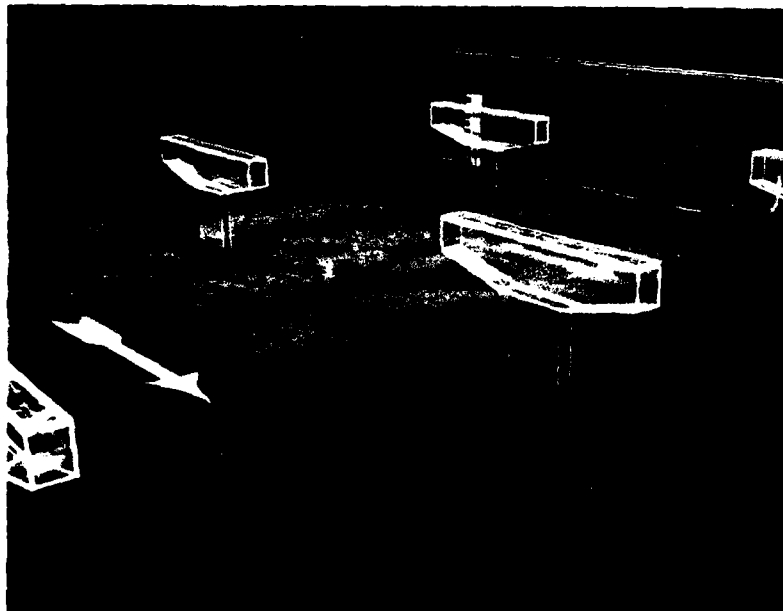
Photograph 34. Scour patterns, railroad bridge pier, partial blockage of channel at bridge. Discharge 100 cfs/foot, approach velocity 10.92 fps, depth 9.16 feet. Duration 1 hr 36 min.



Photograph 35. Scour patterns, six highway bridge piers.
100-year flood, discharge 44 cfs/foot,
velocity 4.50 fps, depth 9.84 feet.
Duration 1 hr 56 min.



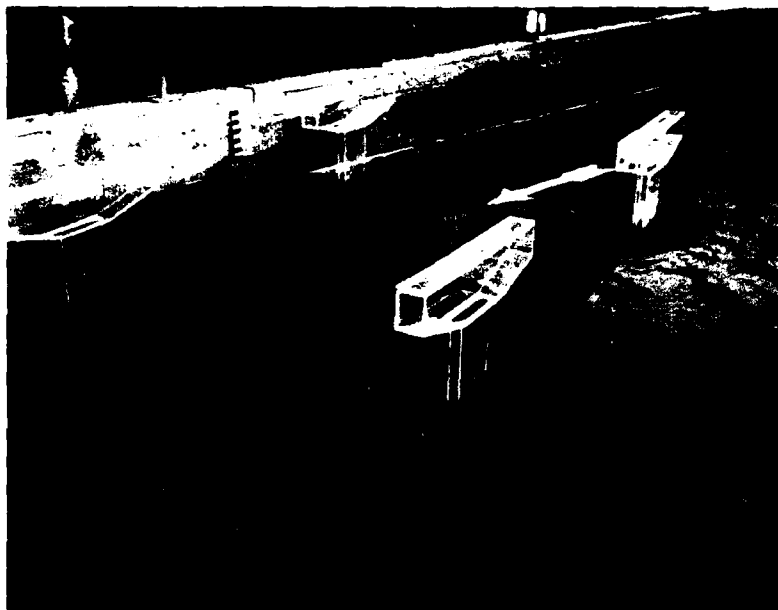
Photograph 36. Scour patterns, six highway bridge piers. Standard project flood, discharge 80 cfs/foot, velocity 6.39 fps, depth 12.60 feet. Duration 1 hr 36 min.



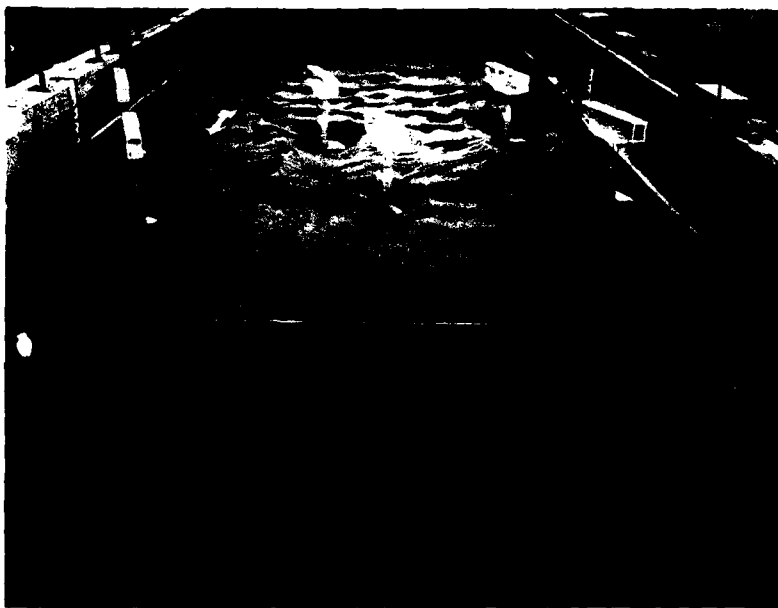
Photograph 37. Scour patterns, six highway bridge piers. Probable maximum flood, discharge 147 cfs/foot, velocity 8.60 fps, depth 17.10 feet. Duration 1 hr 56 min.



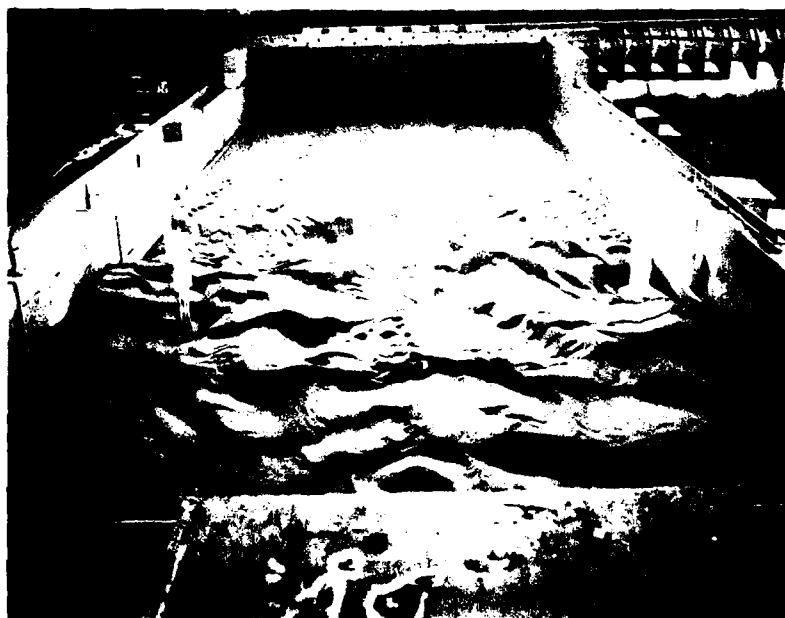
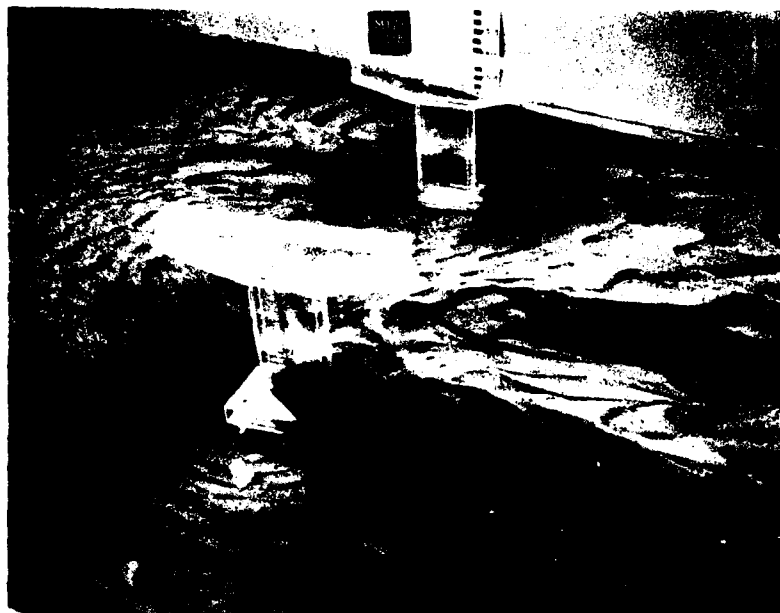
Photograph 38. Scour patterns, six highway piers, 15-degree approach flow. 100-year flood, discharge 44 cfs/foot, velocity 4.50 fps, depth 9.84 feet. Duration 1 hr 56 min.



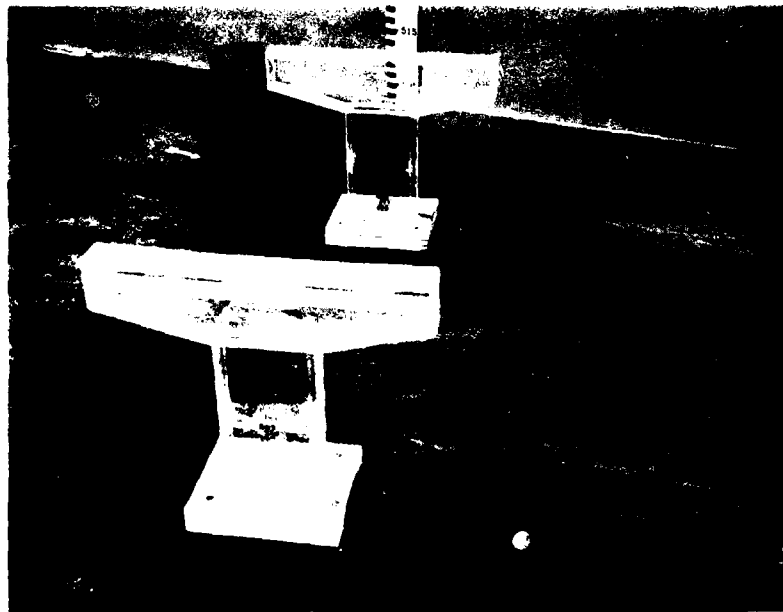
Photograph 39. Scour patterns, six highway piers, 15-degree approach flow. Standard project flood, discharge 80 cfs/foot, velocity 6.39 fps, depth 12.60 feet. Duration 1 hr 56 min.



Photograph 40. Scour patterns, six highway piers, 15-degree approach flow. Probable maximum flood, discharge 147 cfs/foot, velocity 8.60 fps, depth 17.10 feet. Duration 2 hrs 15 min.



Photograph 41. Scour patterns, six highway piers,
 15-degree approach flow. Timber debris
 on three upstream piers. Standard
 project flood, discharge 80 cfs/foot,
 velocity 6.39 fps, depth 12.60 feet.
 Duration 1 hr 56 min.



Photograph 42. Scour patterns, six highway piers,
 15-degree approach flow. Timber debris
 on three upstream piers. Probable
 maximum flood, discharge 147 cfs/foot,
 velocity 8.60 fps, depth 17.10 feet.
 Duration 1 hr 56 min.

AD-A147 036

MOOSE CREEK DAM OUTLET WORKS AND DIVERSION CHANNEL
CHENA RIVER LAKES PROJ..(U) ARMY ENGINEER DIV NORTH
PACIFIC BONNEVILLE OR DIV HYDRAULIC L.. SEP 84

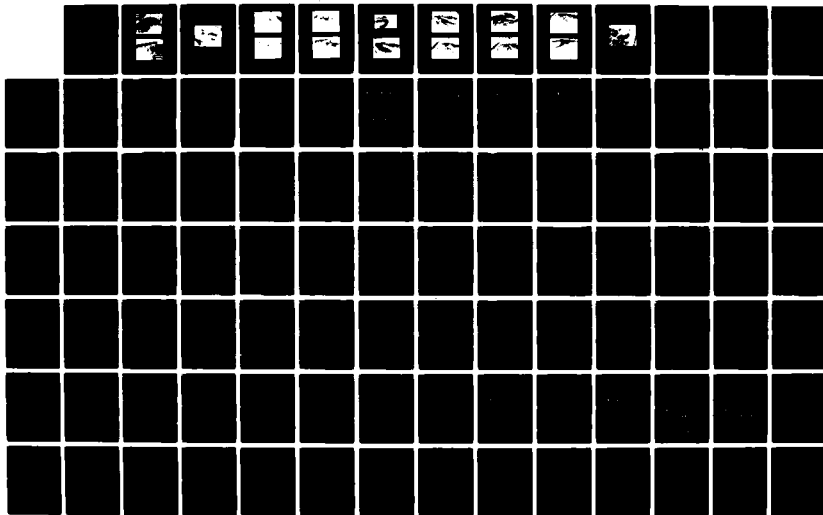
2/3

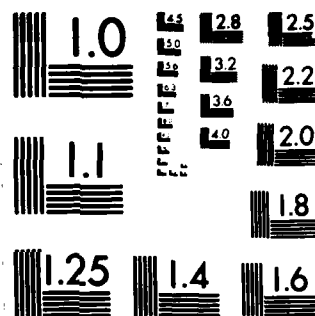
UNCLASSIFIED

TR-158-1

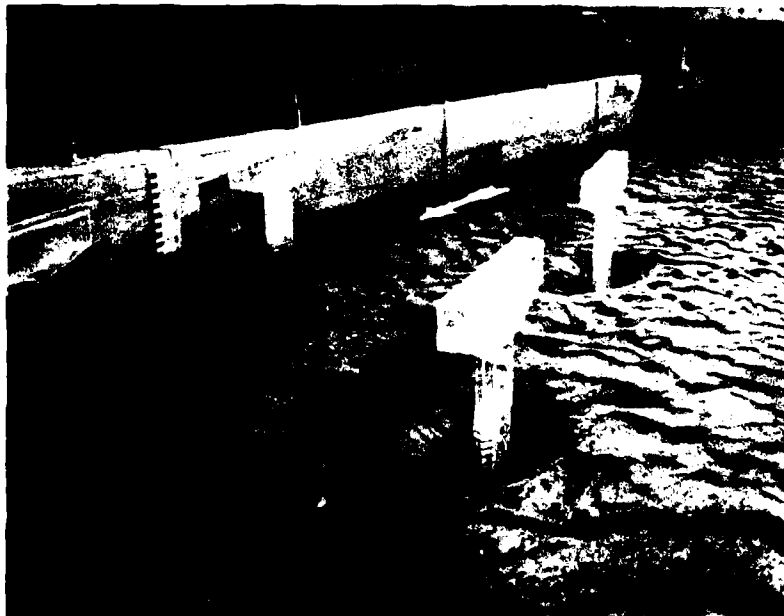
F/G 13/2

NL

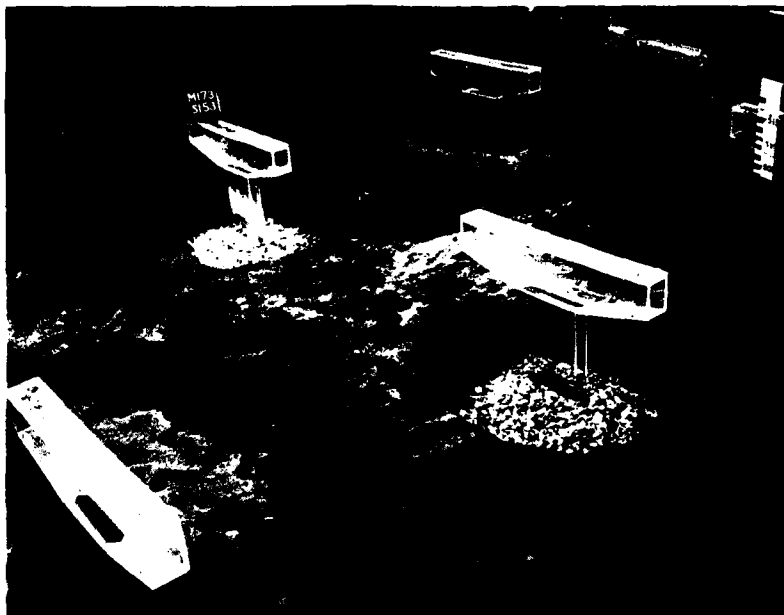




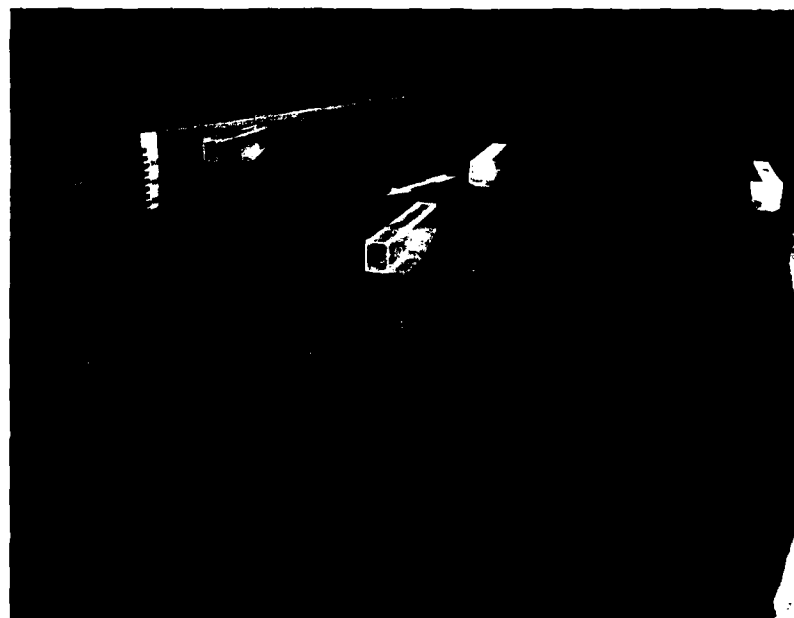
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A



Photograph 43. Scour patterns, six highway piers, 15-degree approach flow. Fine bed material. Standard project flood, discharge 80 cfs/foot, velocity 6.39 fps, depth 12.60 feet. Duration 1 hr 56 min.



Photograph 44. Type I rock protection placed to elevation 498.0 at two highway piers prior to backfilling and molding channel bed to elevation 502.0.



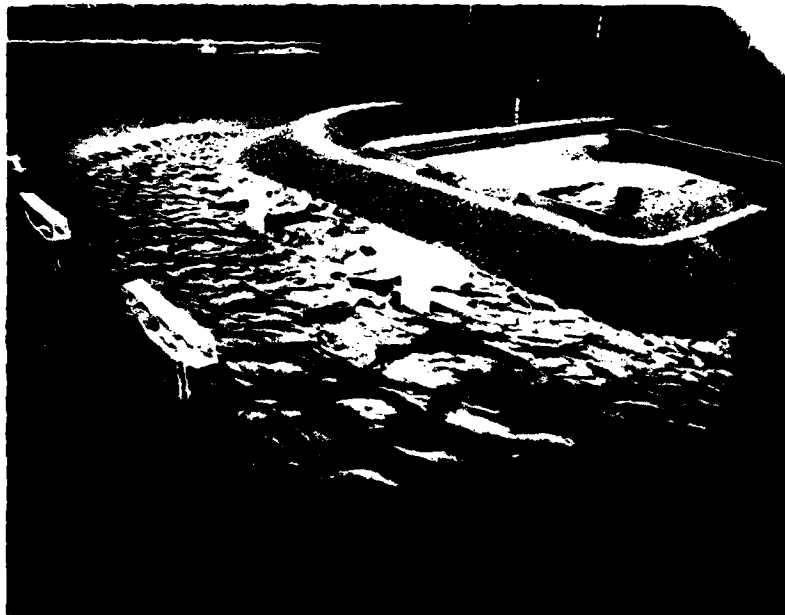
Photograph 45. Scour patterns, six highway piers, 15-degree approach flow. Type I rock protection at two center piers. Standard project flood, discharge 80 cfs/foot, velocity 6.39 fps, depth 12.60 feet. Duration 1 hr 56 min.



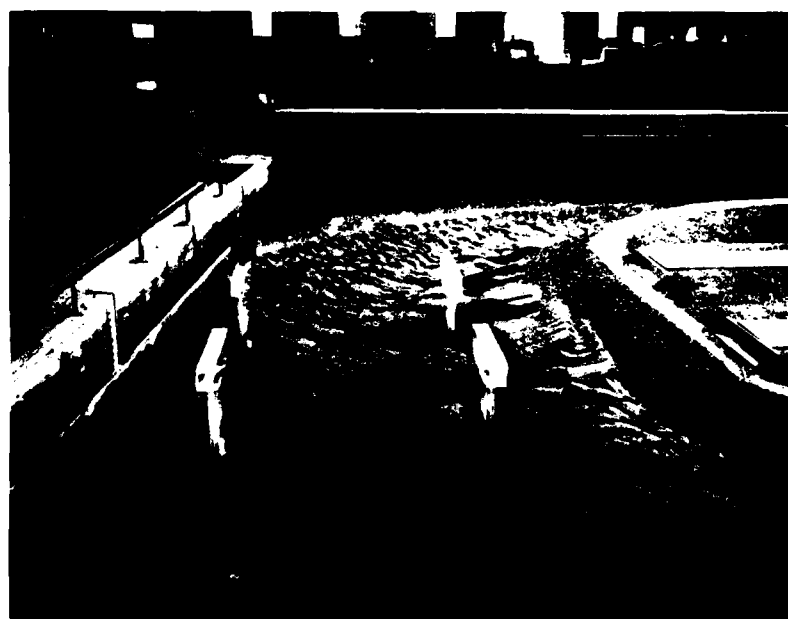
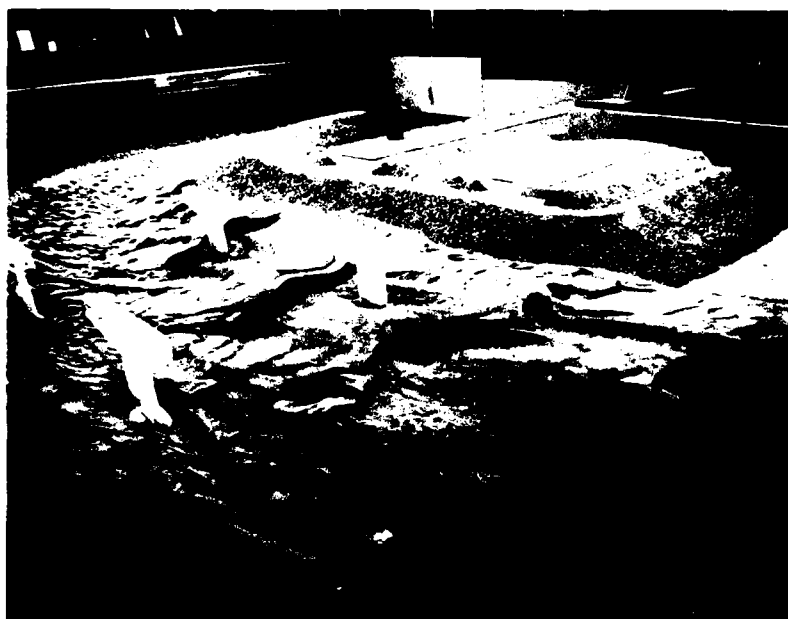
Photograph 46. Scour patterns, six highway piers, 15-degree approach flow. Clear-water scour. Standard project flood, discharge 80 cfs/foot, velocity 6.39 fps, depth 12.60 feet. Duration 1 hr 56 min.



Photograph 47. General views of abutment scour model. Bed material, $D_{50} = 0.43$ mm (6.45 mm prototype). Type II riprap on abutment and training dike.



Photograph 48. Scour pattern, 100-year flood.
Duration of test, 7.75 hrs.



Photograph 49. Scour pattern, standard project flood.
Duration of test, 7.75 hrs.



M 173 A25

Photograph 50. Scour pattern, probable maximum flood.
Duration of test, 7.75 hrs.



Photograph 51. Debris simulated on piers during standard project flood (shown at base of piers after test).

2

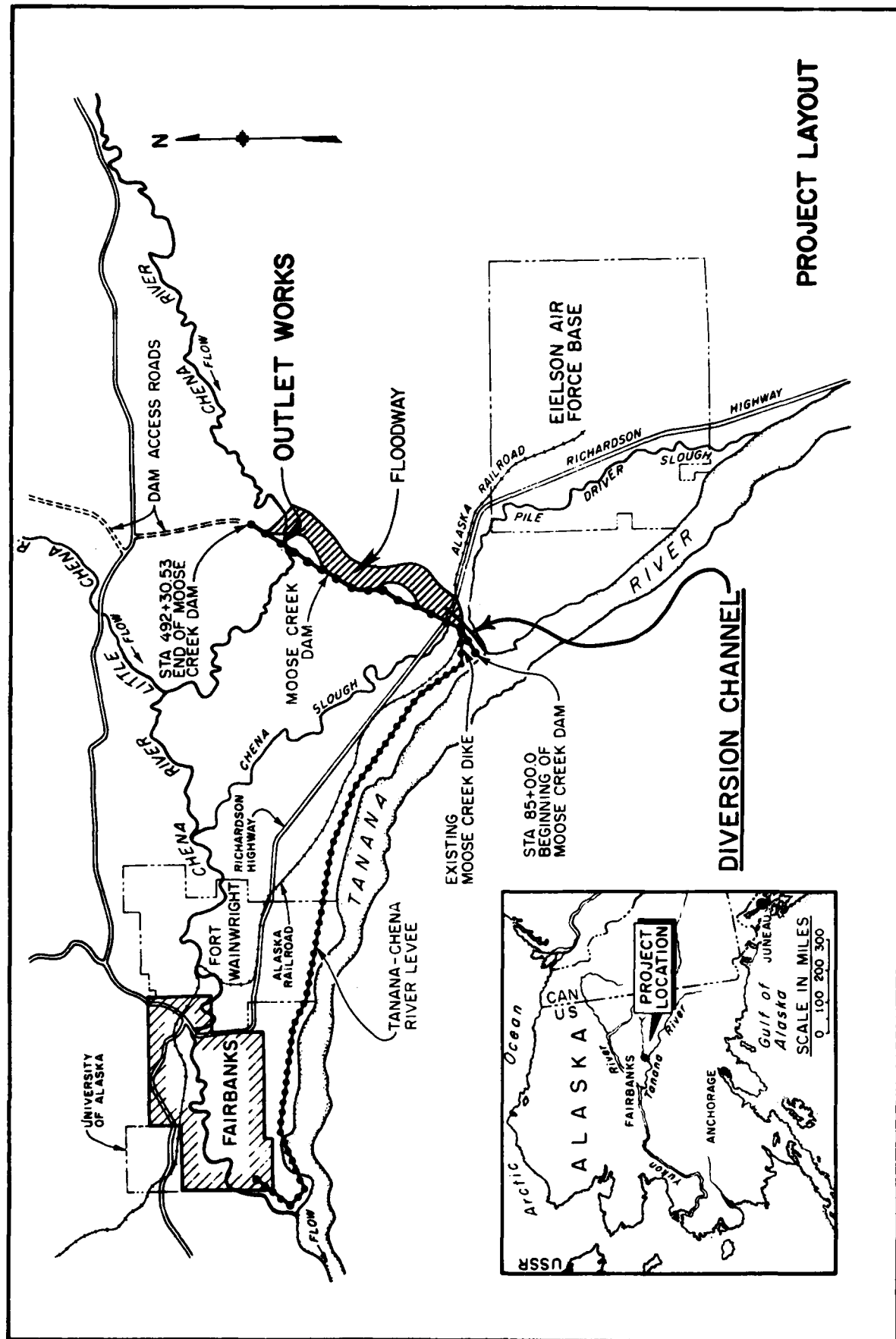
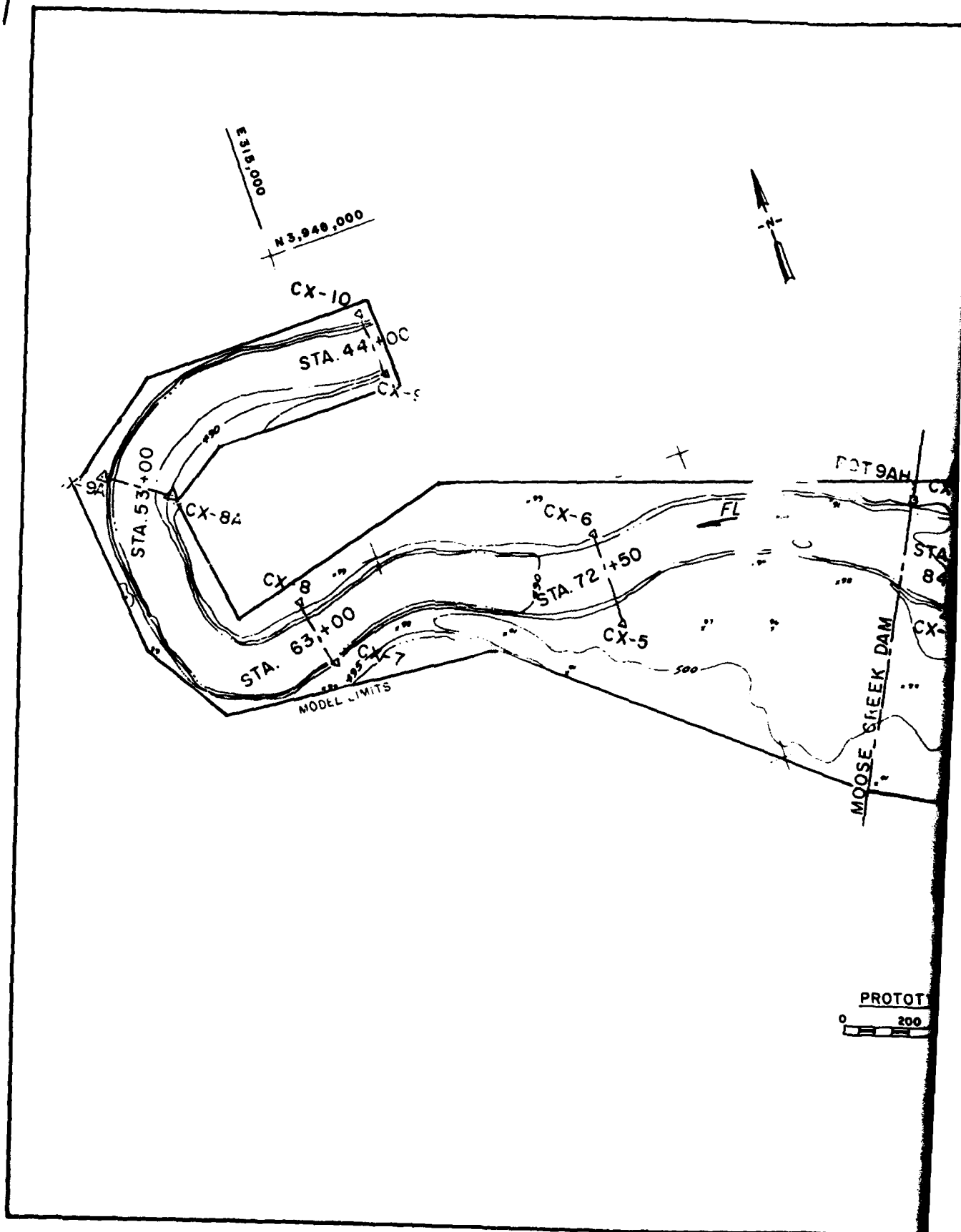
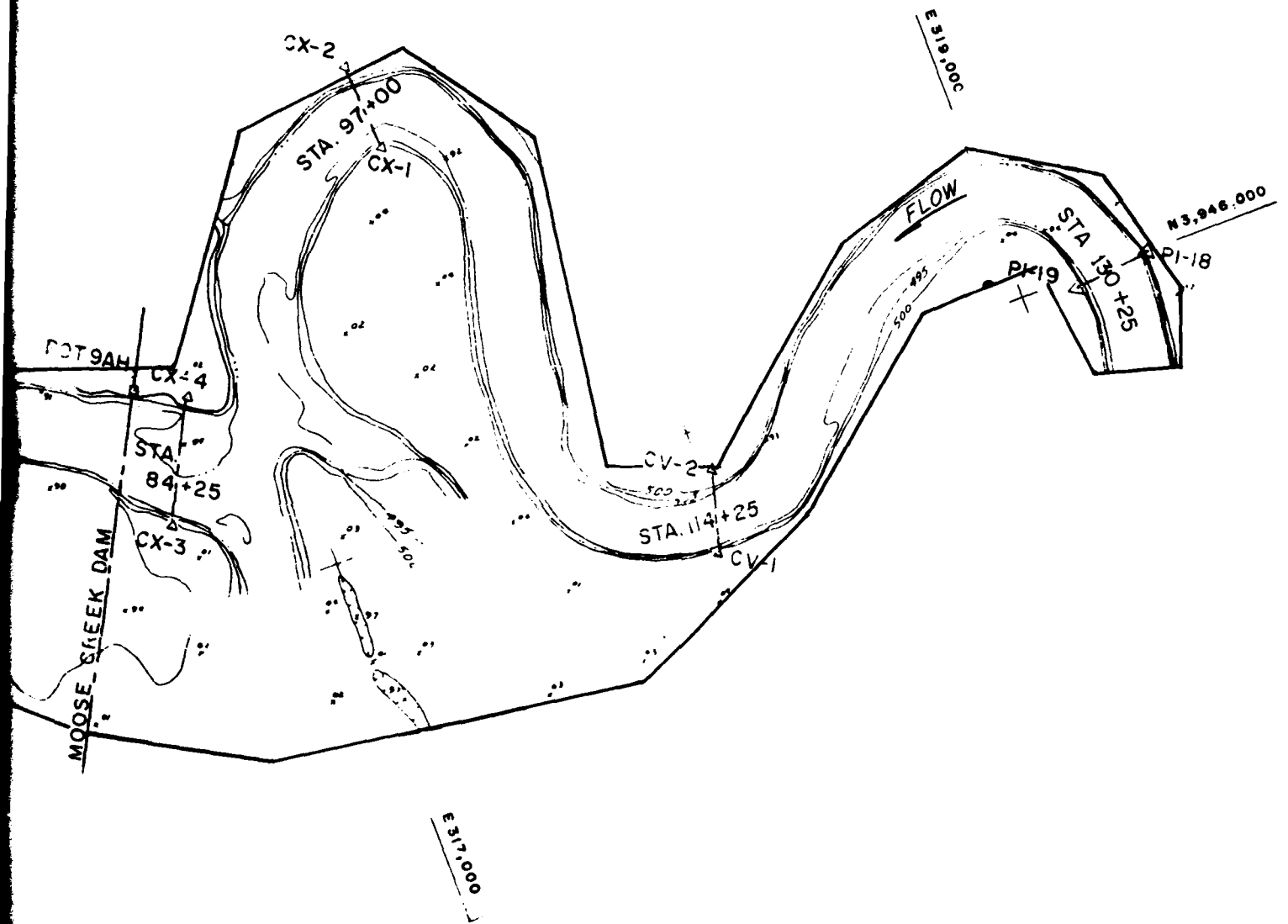


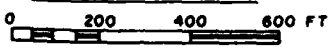
PLATE 1



2



PROTOTYPE SCALE

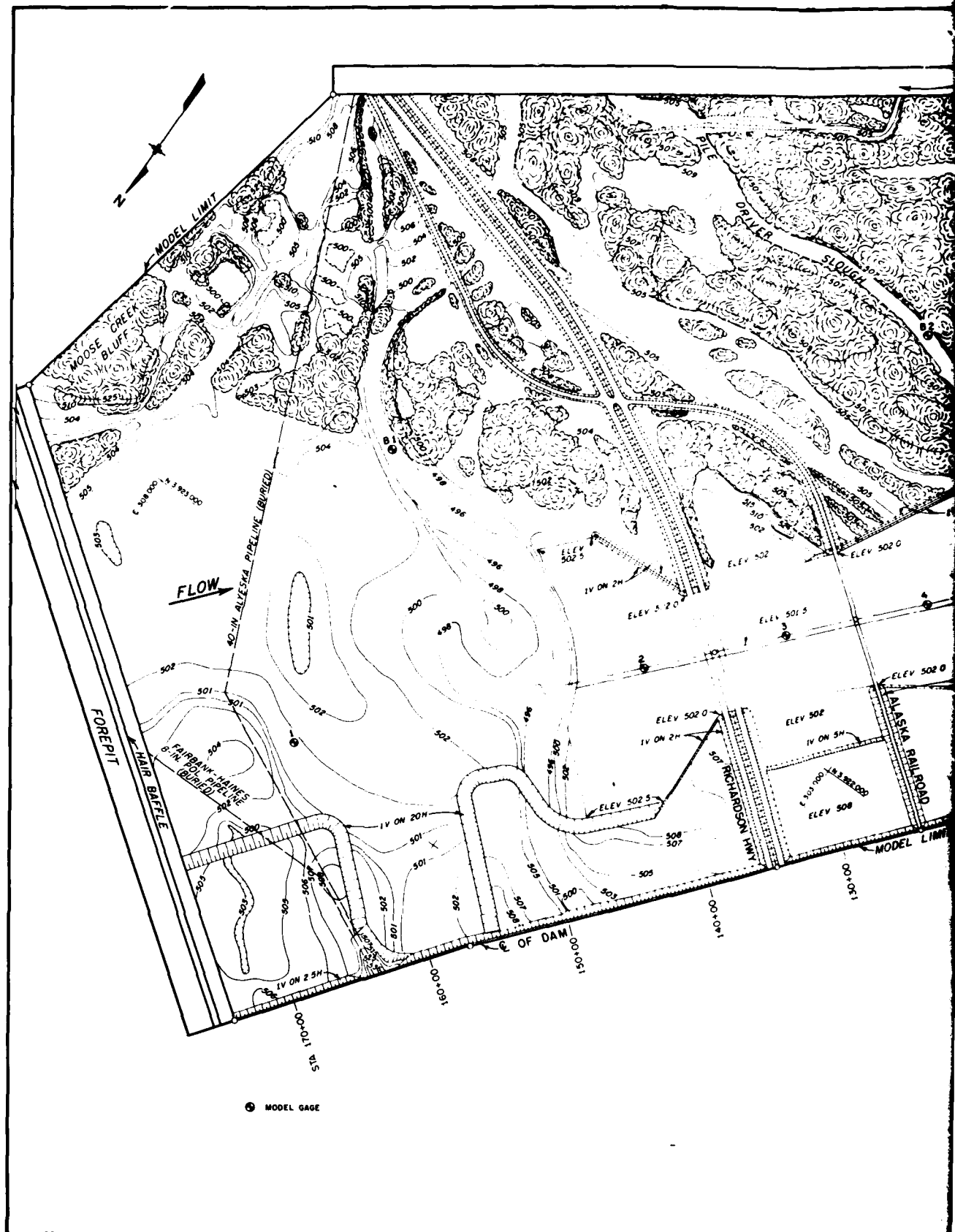


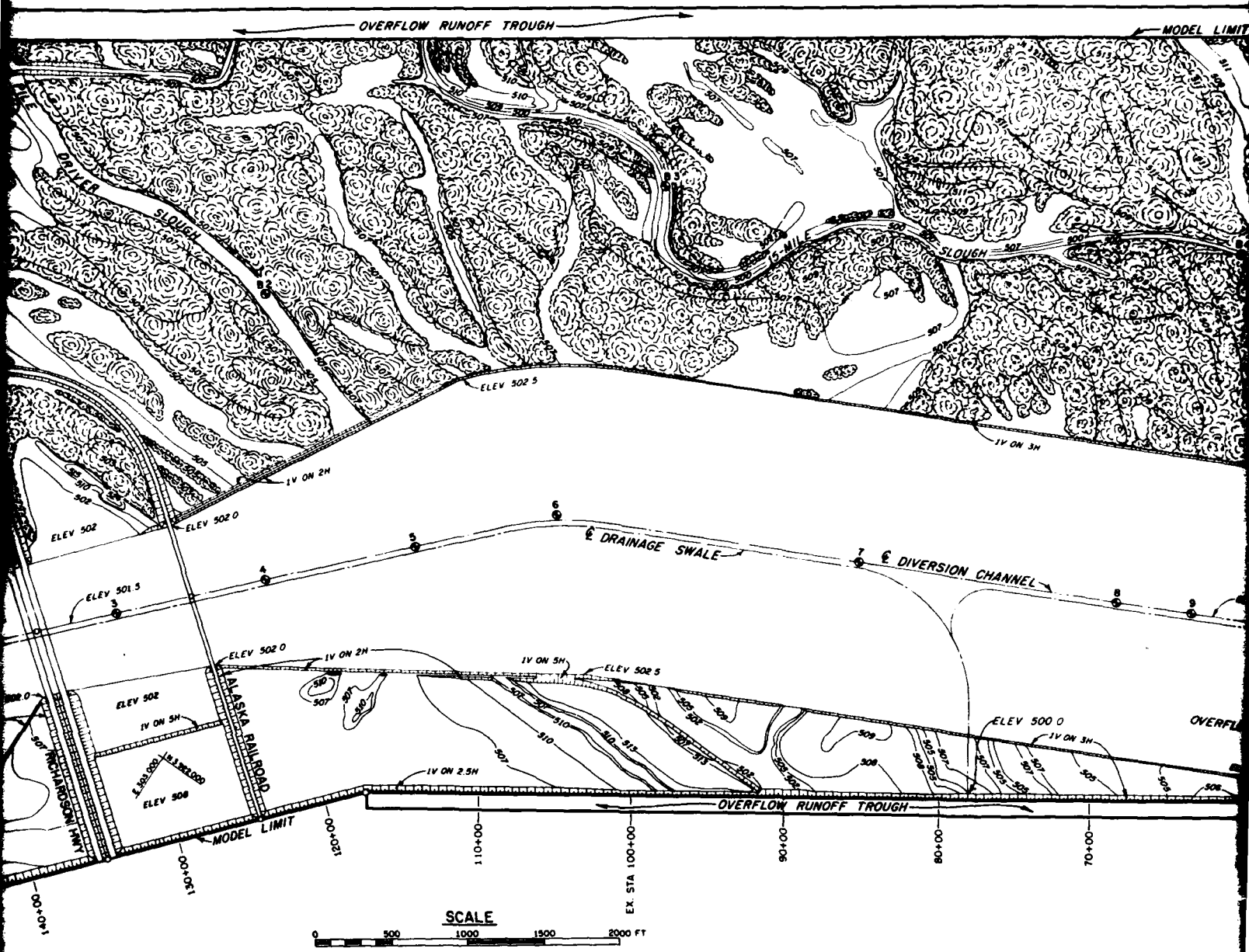
OUTLET CHANNEL MODEL LAYOUT

PLATE 2

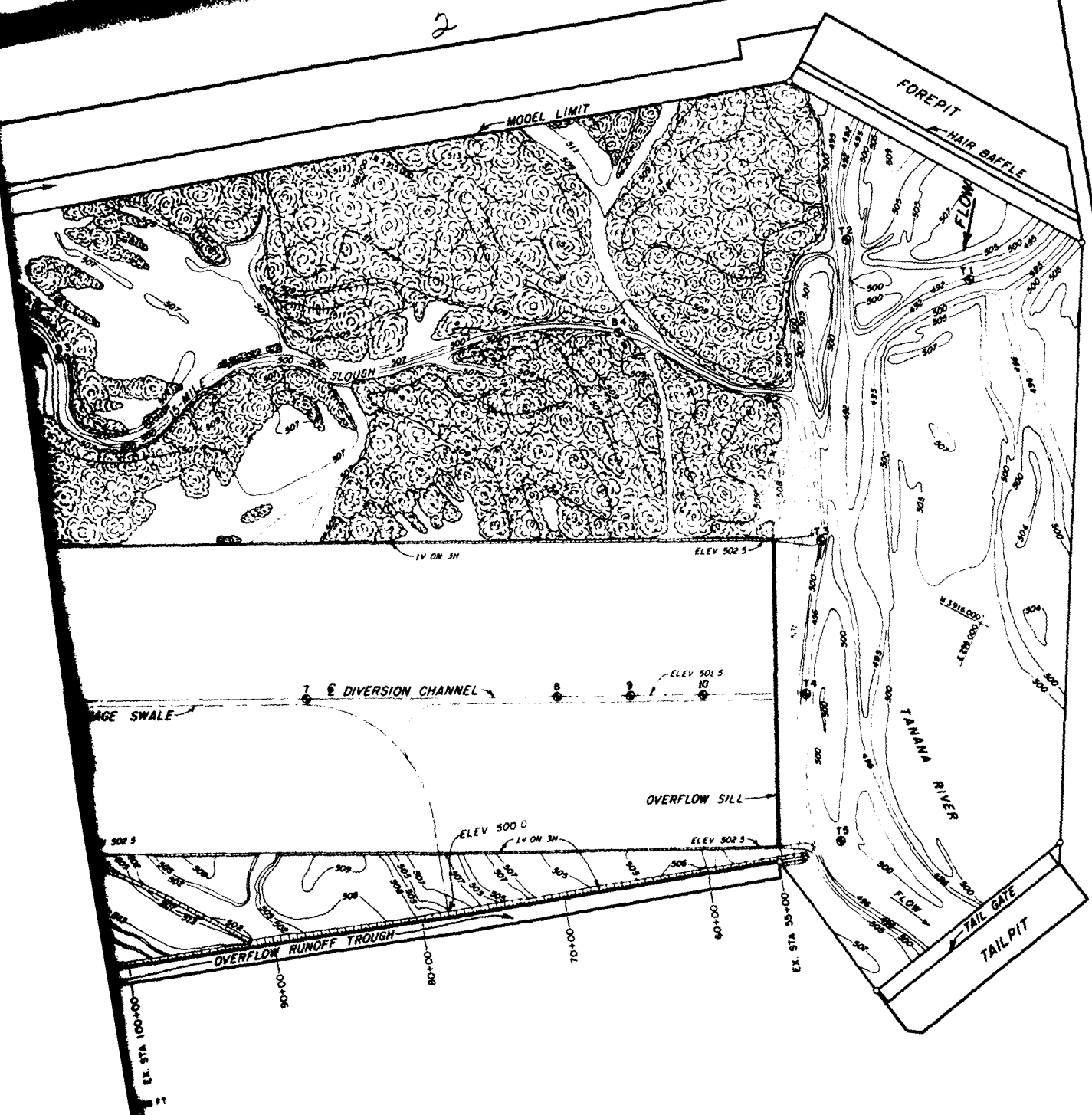
1

2





2

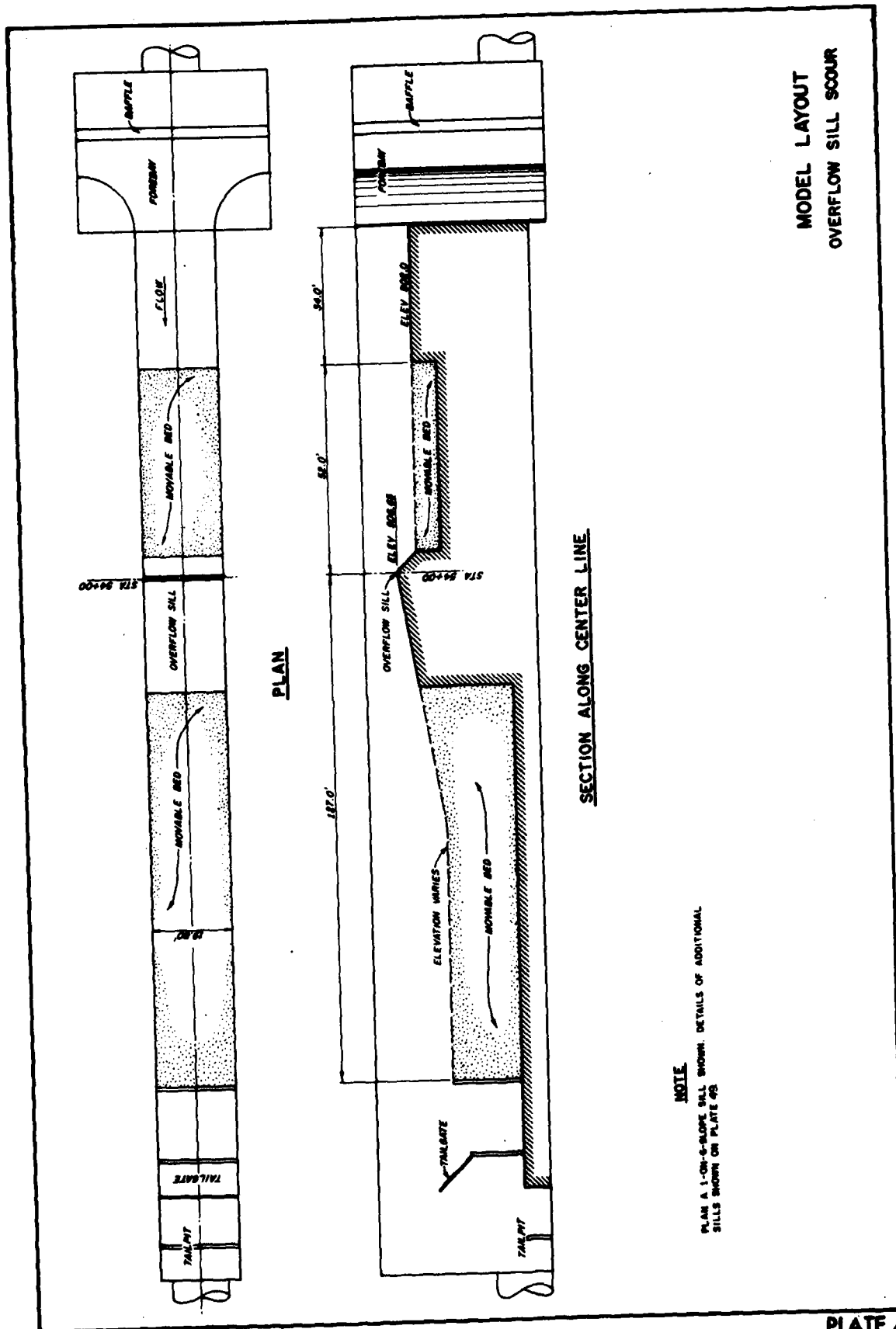


DIVERSION CHANNEL MODEL LAYOUT

PLATE 3

3

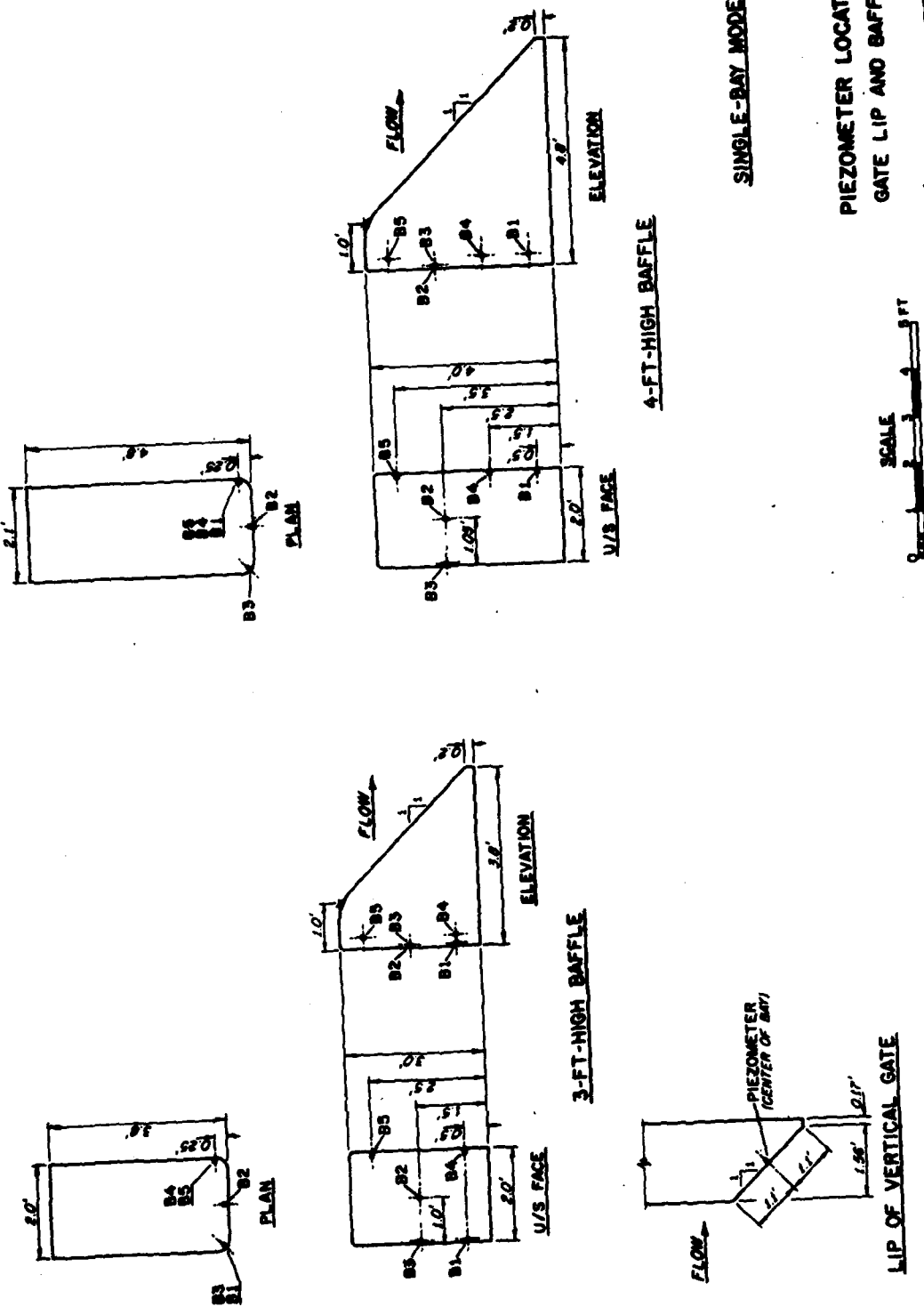
2



NOTE
 PLAN A 1-IN-8-SLOPE SILL SHOWN. DETAILS OF ADDITIONAL
 SILLS SHOWN ON PLATE #5

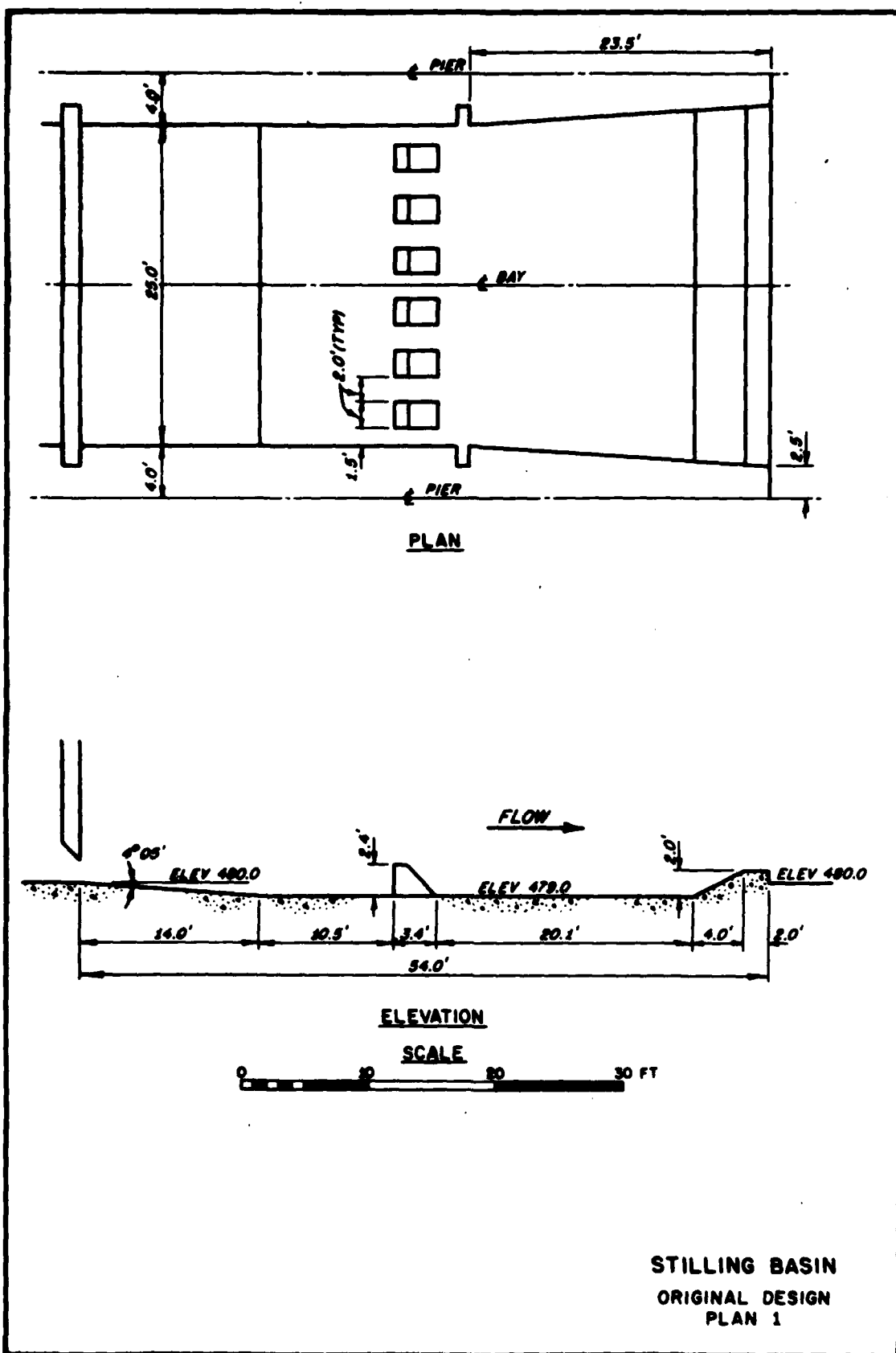
MODEL LAYOUT
OVERFLOW SILL SCOUR

2

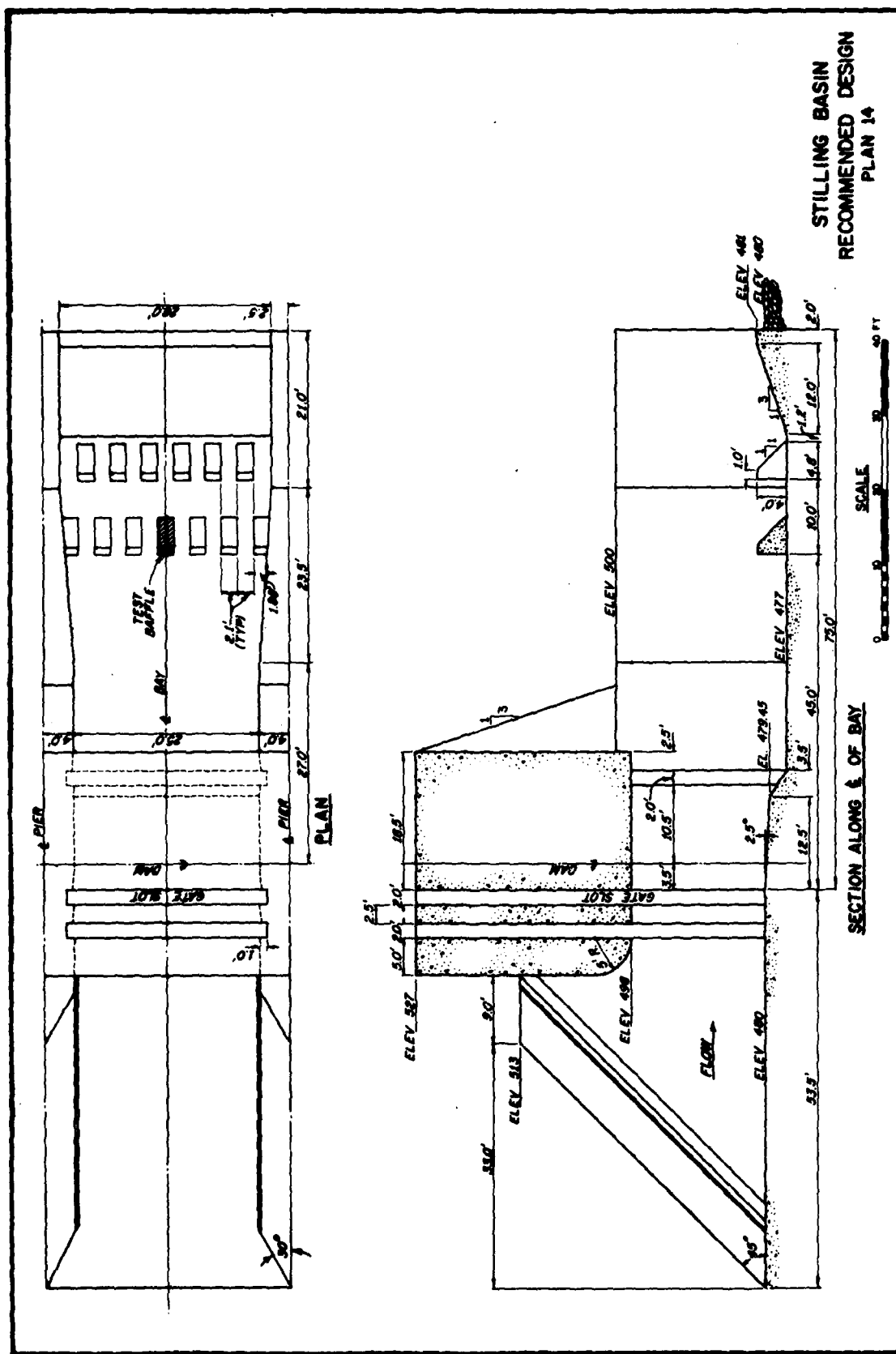


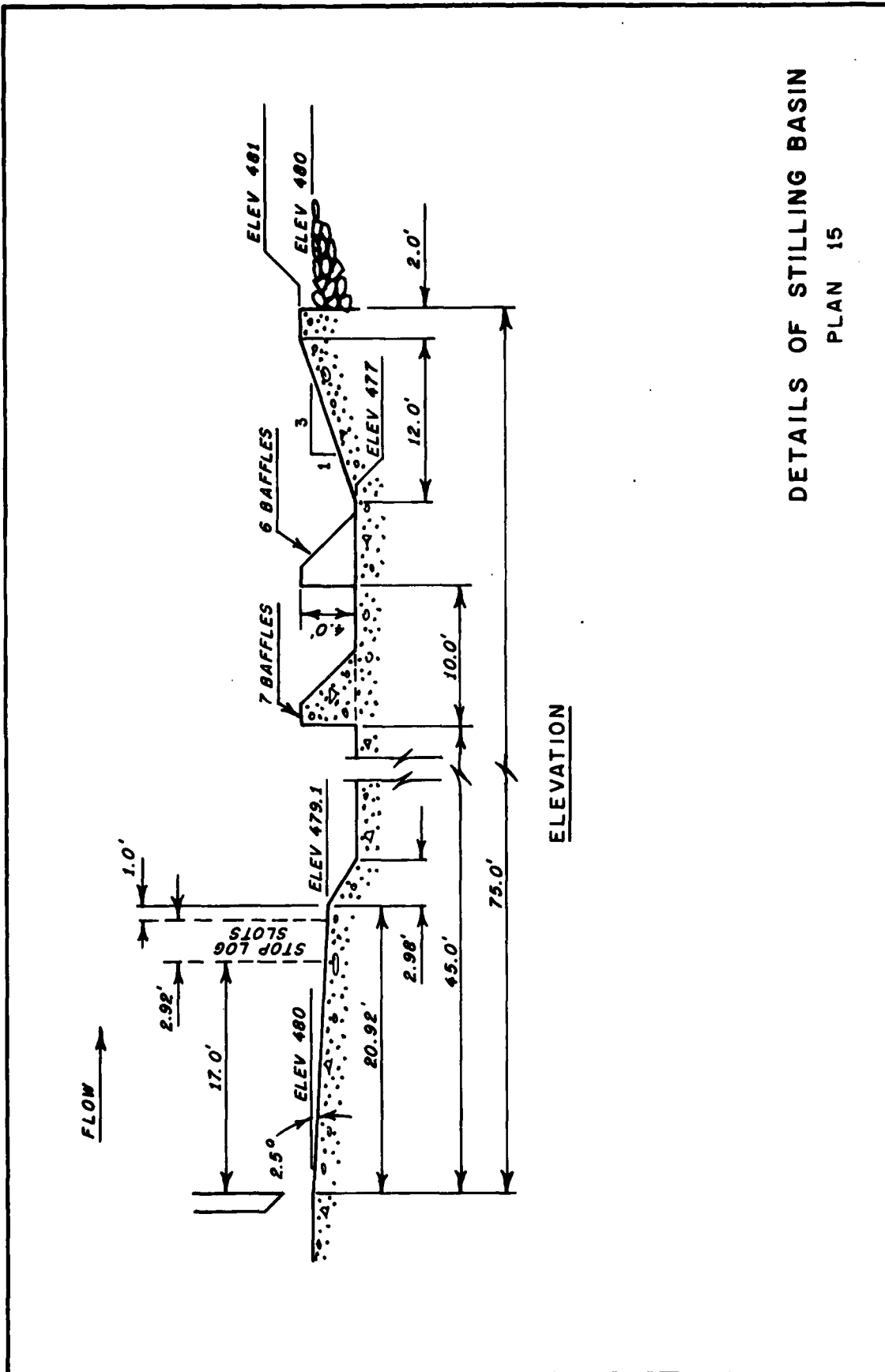
SINGLE-BAY MODEL

PIEZOMETER LOCATIONS
GATE LIP AND BAFFLES



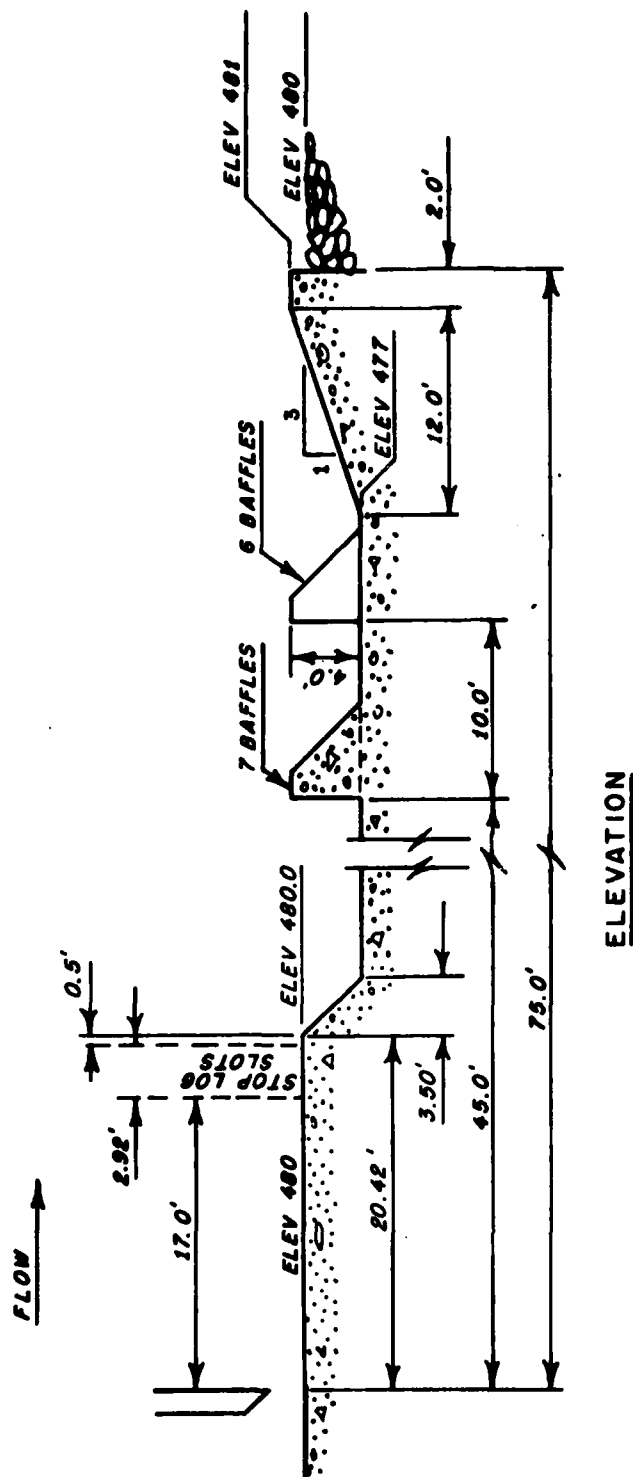
PLANS 12 AND 13





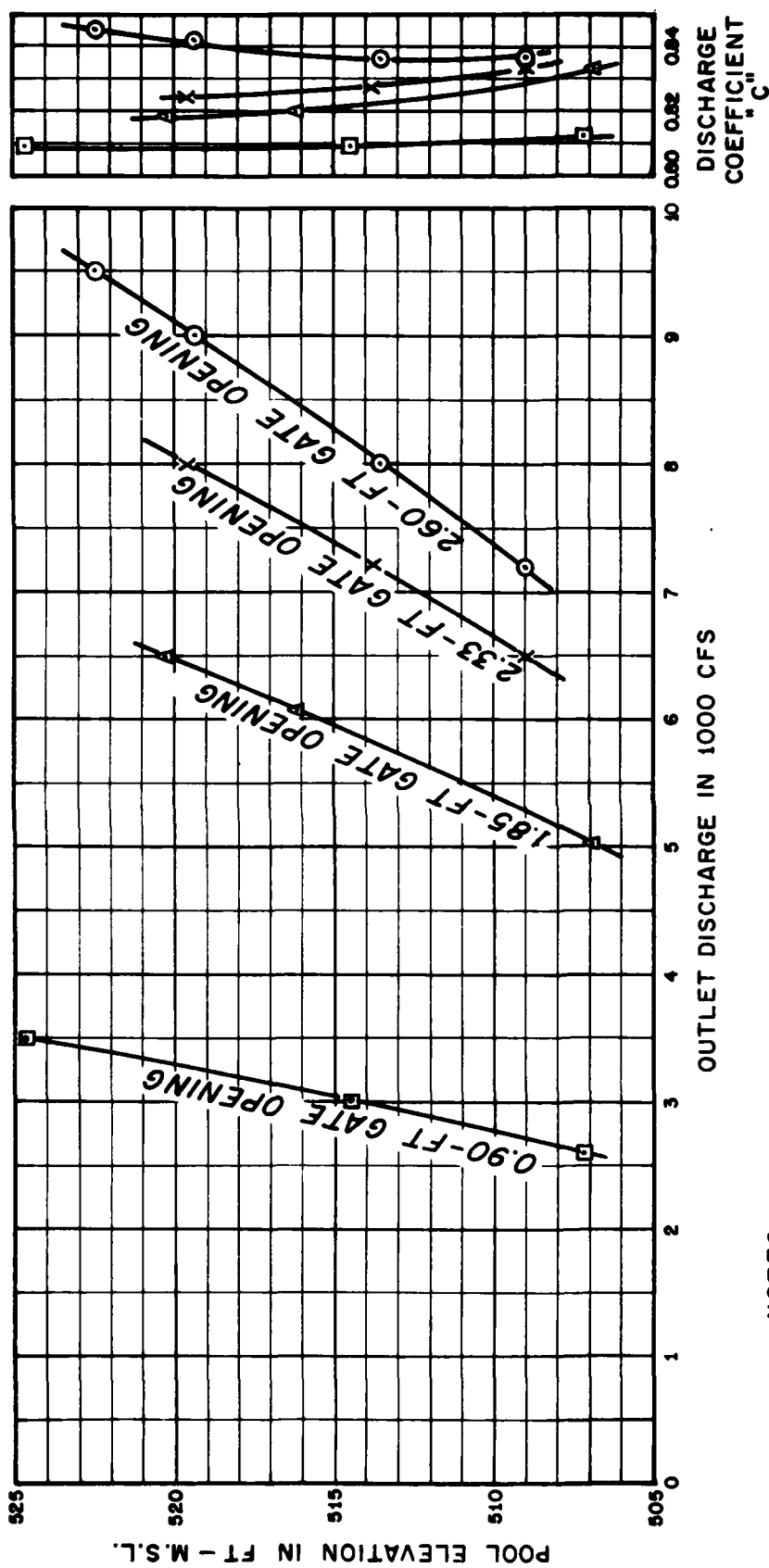
ELEVATION

DETAILS OF STILLING BASIN
PLAN 15



ELEVATION

DETAILS OF STILLING BASIN
PLAN 16 (FINAL DESIGN)

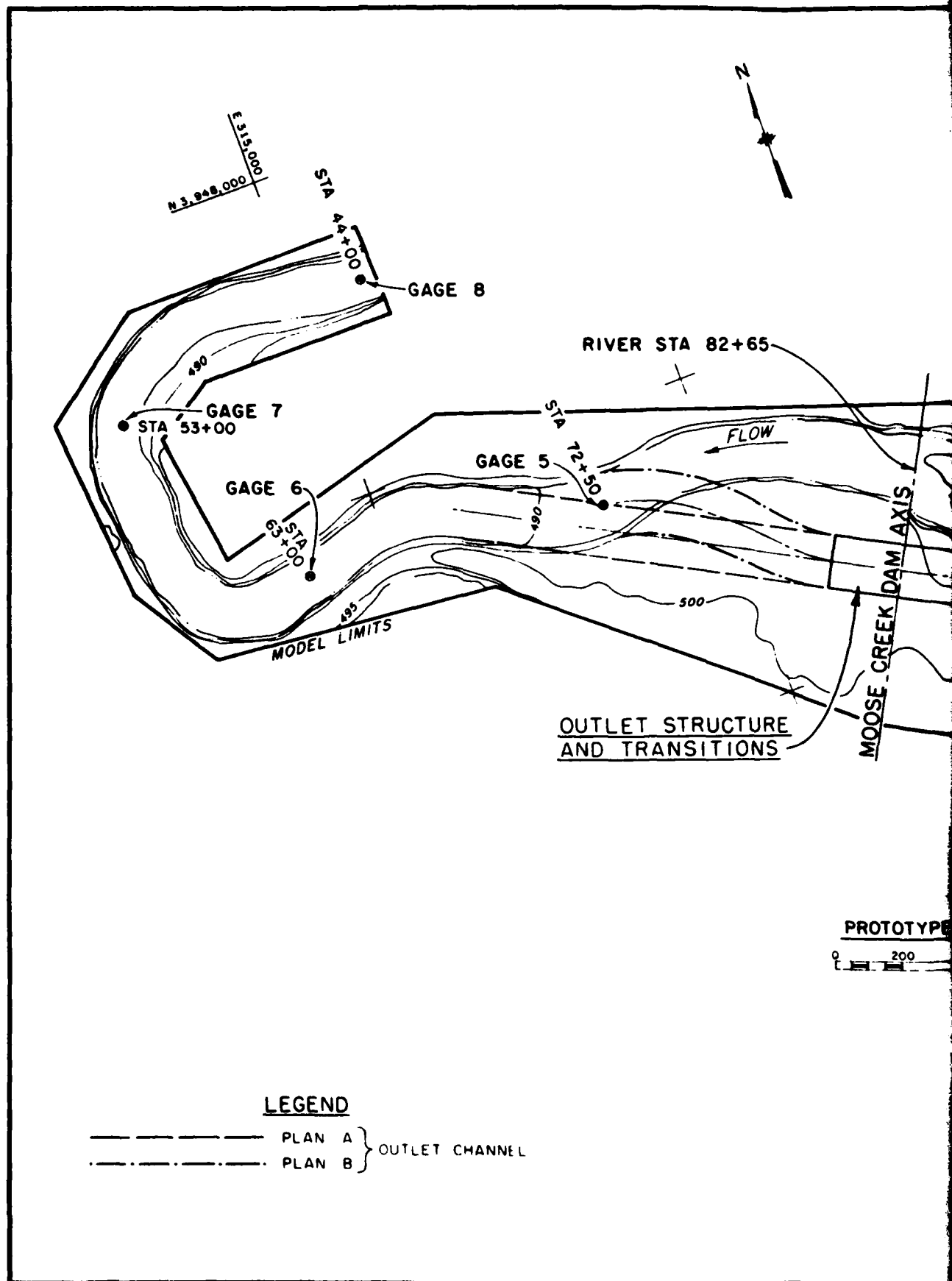


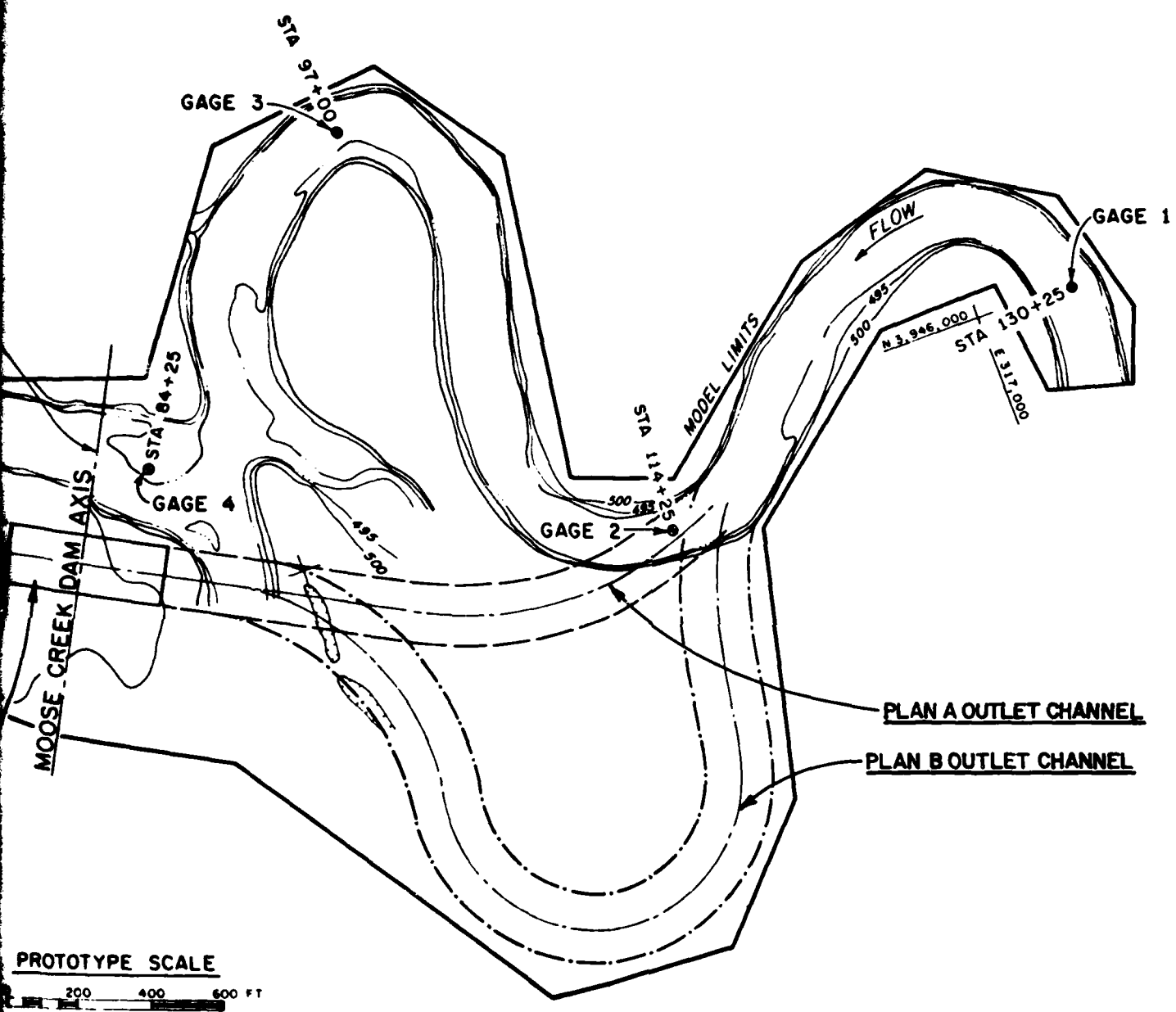
NOTES

1. DATA OBTAINED FROM 1:20 - SCALE SINGLE-BAY MODEL.
2. GATE RATING AND COEFFICIENTS SHOWN IN TABLE C.

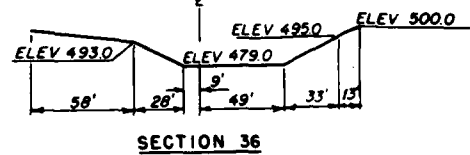
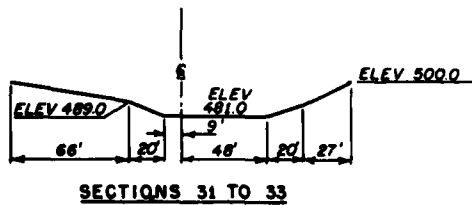
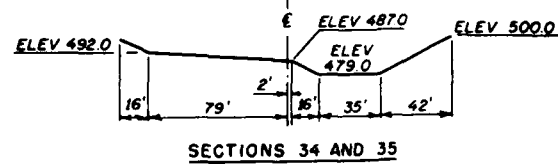
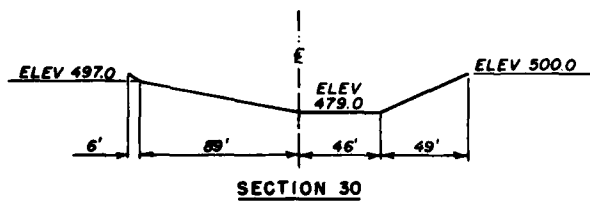
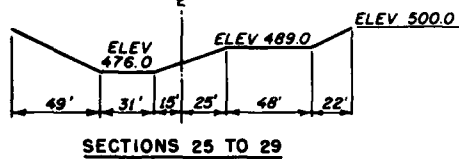
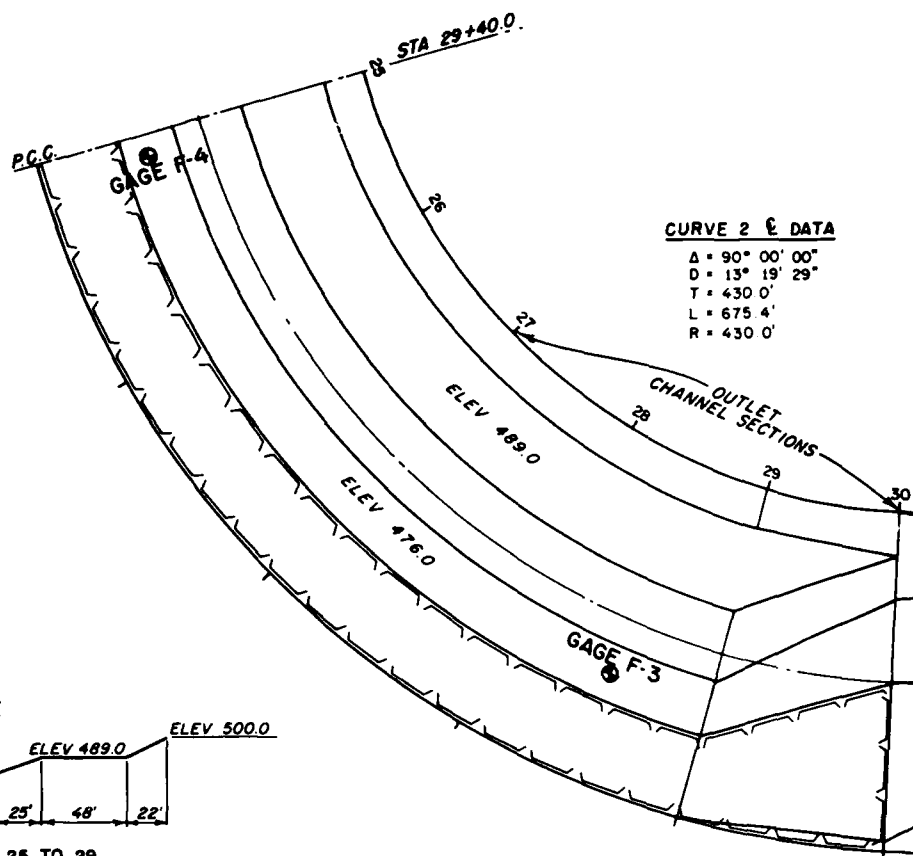
$$Q = CA\sqrt{2g(d_u + h_v - d_d)}$$

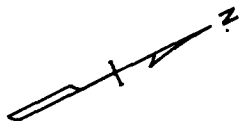
**DISCHARGE RATING
PLAN C OUTLET STRUCTURE**





OUTLET CHANNEL
AND
GAGE LOCATIONS





DATA

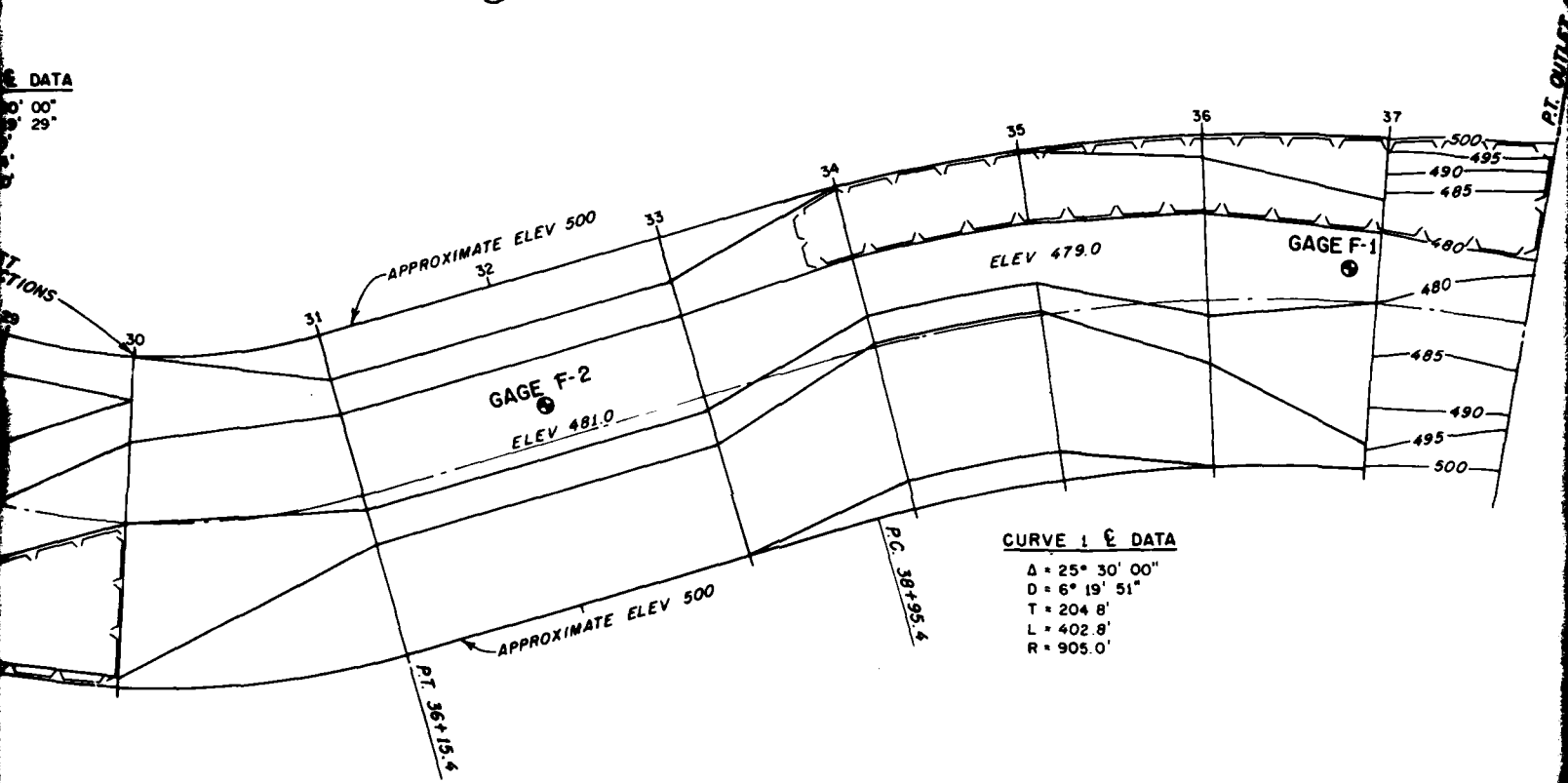
20' 00"
29'

STATIONS

29

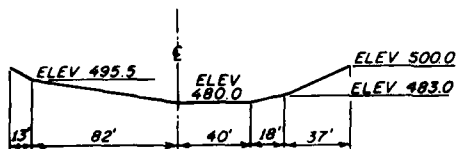
ELEV 500.0
42'

ELEV 500.0
13' 15'



CURVE 1 DATA

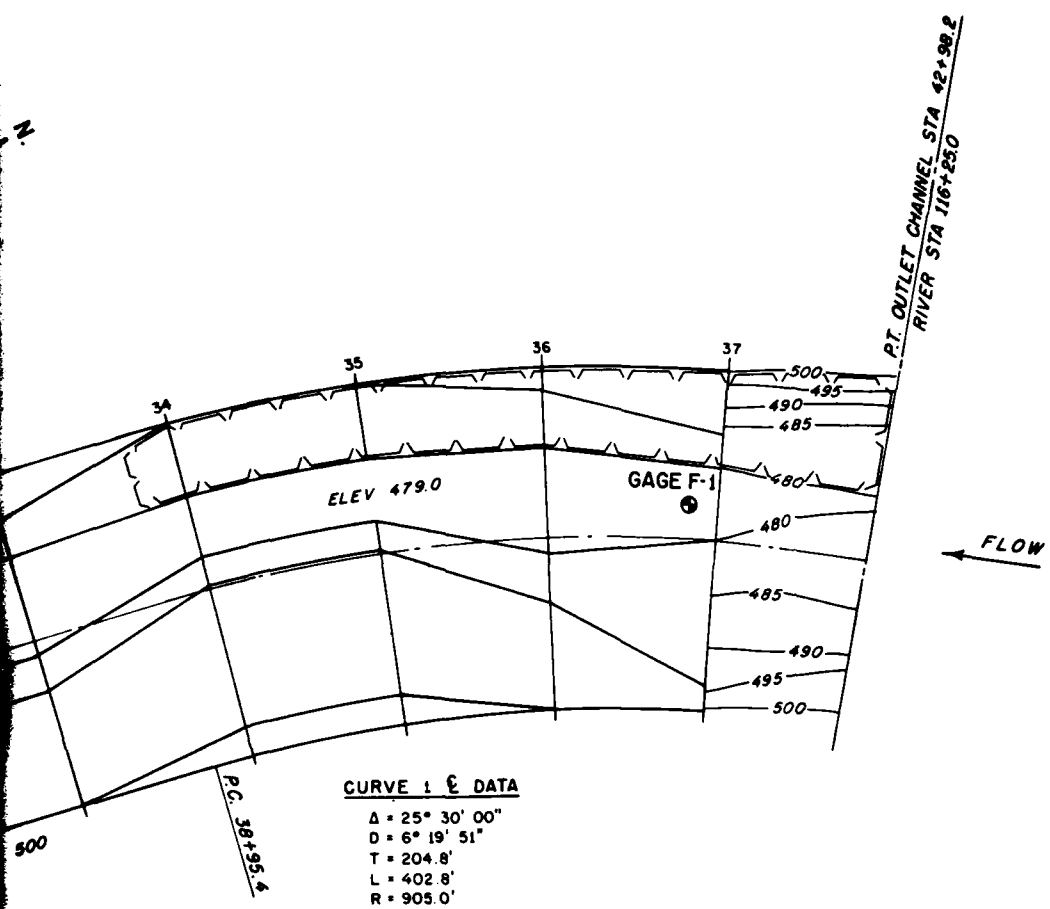
$\Delta = 25^\circ 30' 00''$
 $D = 6^\circ 19' 51''$
 $T = 204.8'$
 $L = 402.8'$
 $R = 905.0'$



SECTION 37

SECTION	STATION
25	29+40.0
26	30+50.0
27	31+60.0
28	32+70.0
29	33+80.0
30	34+80.0
31	36+15.4
32	37+15.4
33	38+15.4
34	39+20.0
35	40+20.0
36	41+20.0
37	42+20.0

2



CURVE 1 & DATA
 $\Delta = 25^\circ 30' 00''$
 $D = 6^\circ 19' 51''$
 $T = 204.8'$
 $L = 402.8'$
 $R = 905.0'$

SECTION	STATION
25	29+40.0
26	30+50.0
27	31+60.0
28	32+70.0
29	33+80.0
30	34+80.0
31	36+15.4
32	37+15.4
33	38+15.4
34	39+20.0
35	40+20.0
36	41+20.0
37	42+20.0

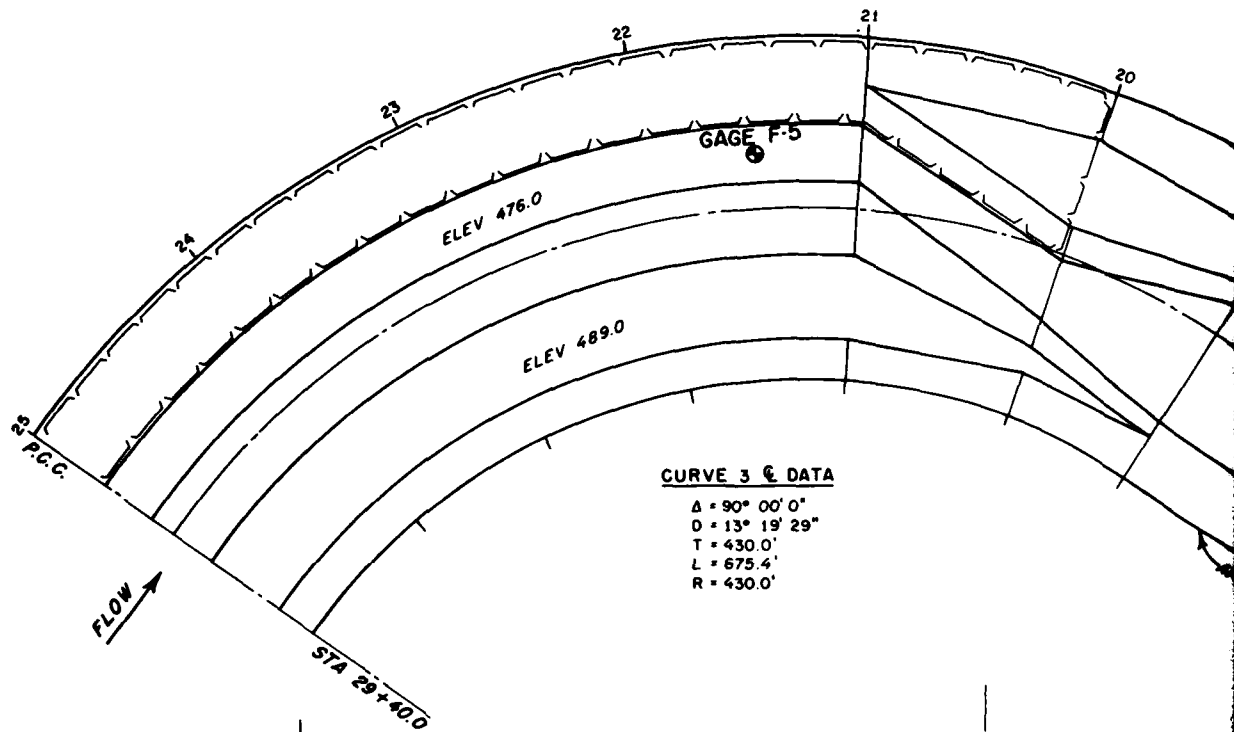
ELEV 500.0
 ELEV 483.0

OUTLET CHANNEL DETAILS
PLAN B
 STATION 29+40.0 TO 42+98.2

2

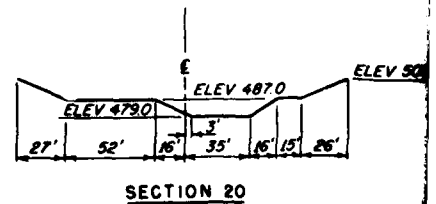
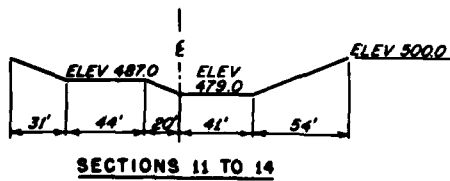
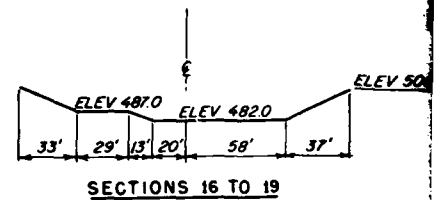
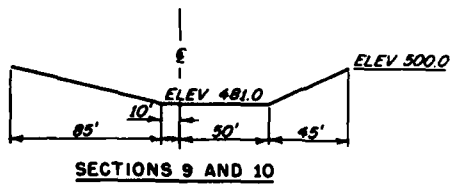
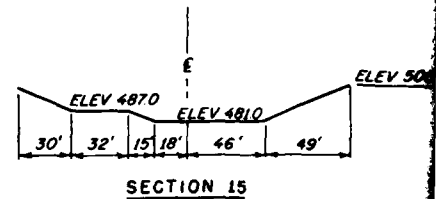
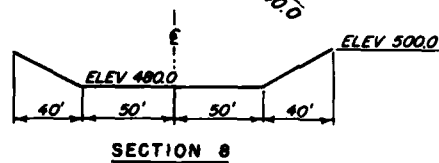
1

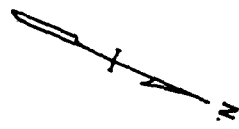
3



CURVE 3 & DATA

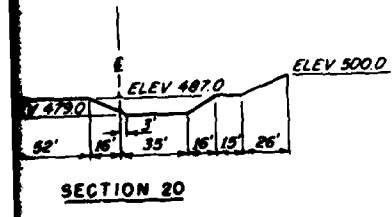
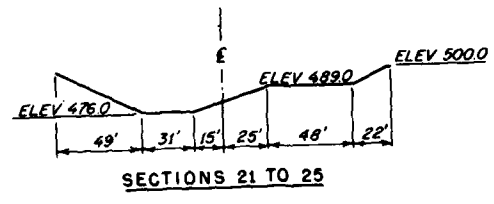
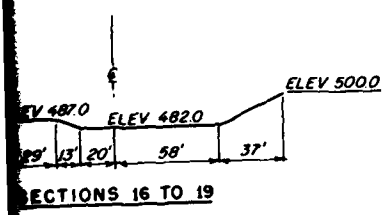
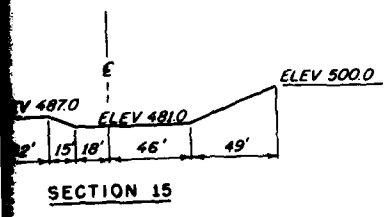
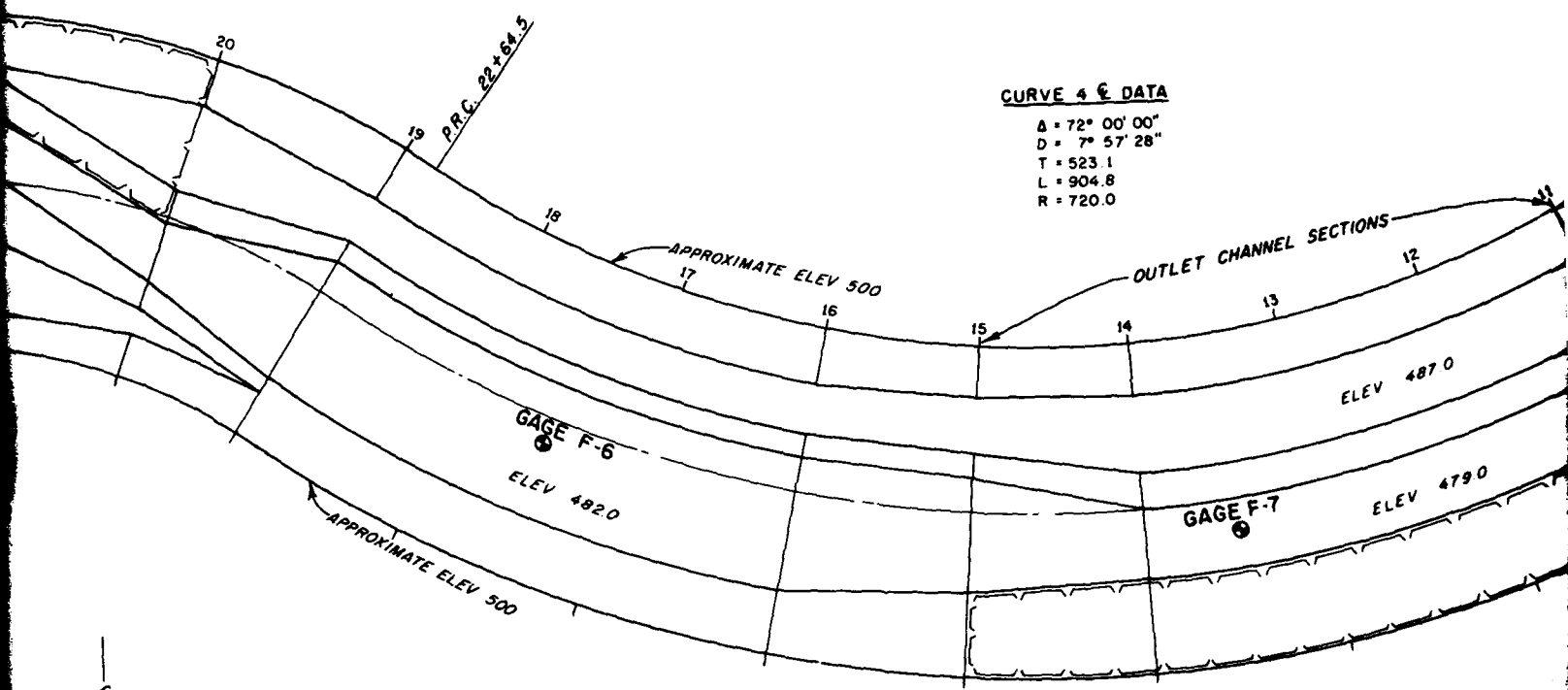
$\Delta = 90^\circ 00' 0''$
 $D = 13^\circ 19' 29''$
 $T = 430.0'$
 $L = 675.4'$
 $R = 430.0'$





CURVE 4 & DATA

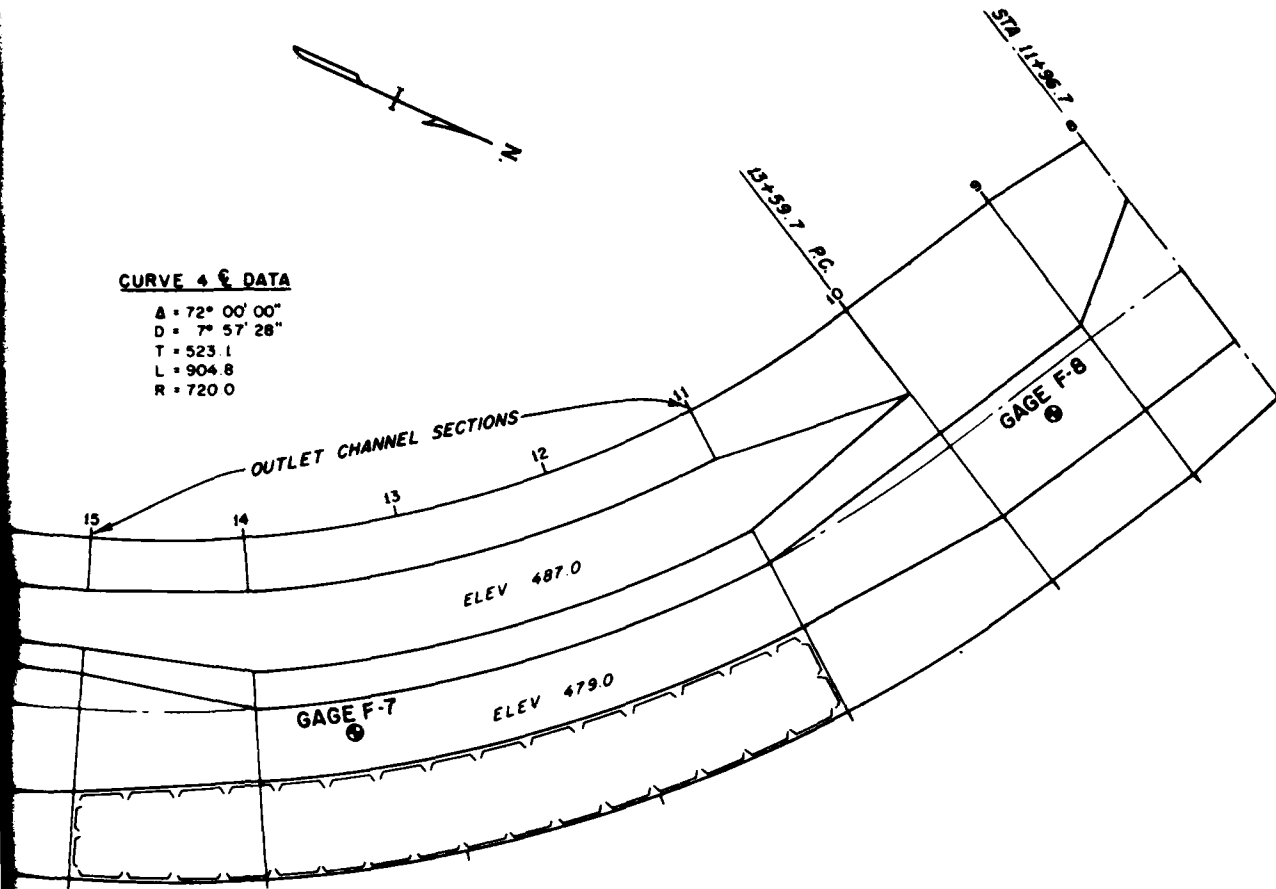
$\Delta = 72^\circ 00' 00''$
 $D = 7^\circ 57' 28''$
 $T = 523.1$
 $L = 904.8$
 $R = 720.0$



SECTION	STATION
8	11+96.7
9	12+59.7
10	13+59.7
11	14+80.0
12	15+80.0
13	16+80.0
14	17+80.0
15	18+80.0
16	19+80.0
17	20+80.0
18	21+80.0
19	22+80.0
20	23+80.0
21	25+00.0
22	26+10.0
23	27+20.0
24	28+30.0
25	29+40.0

CURVE 4 & DATA

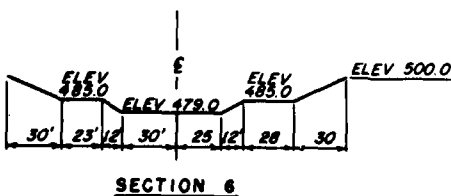
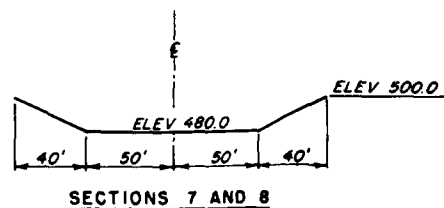
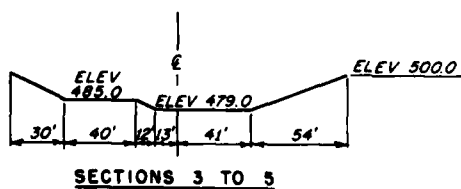
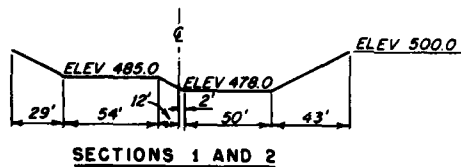
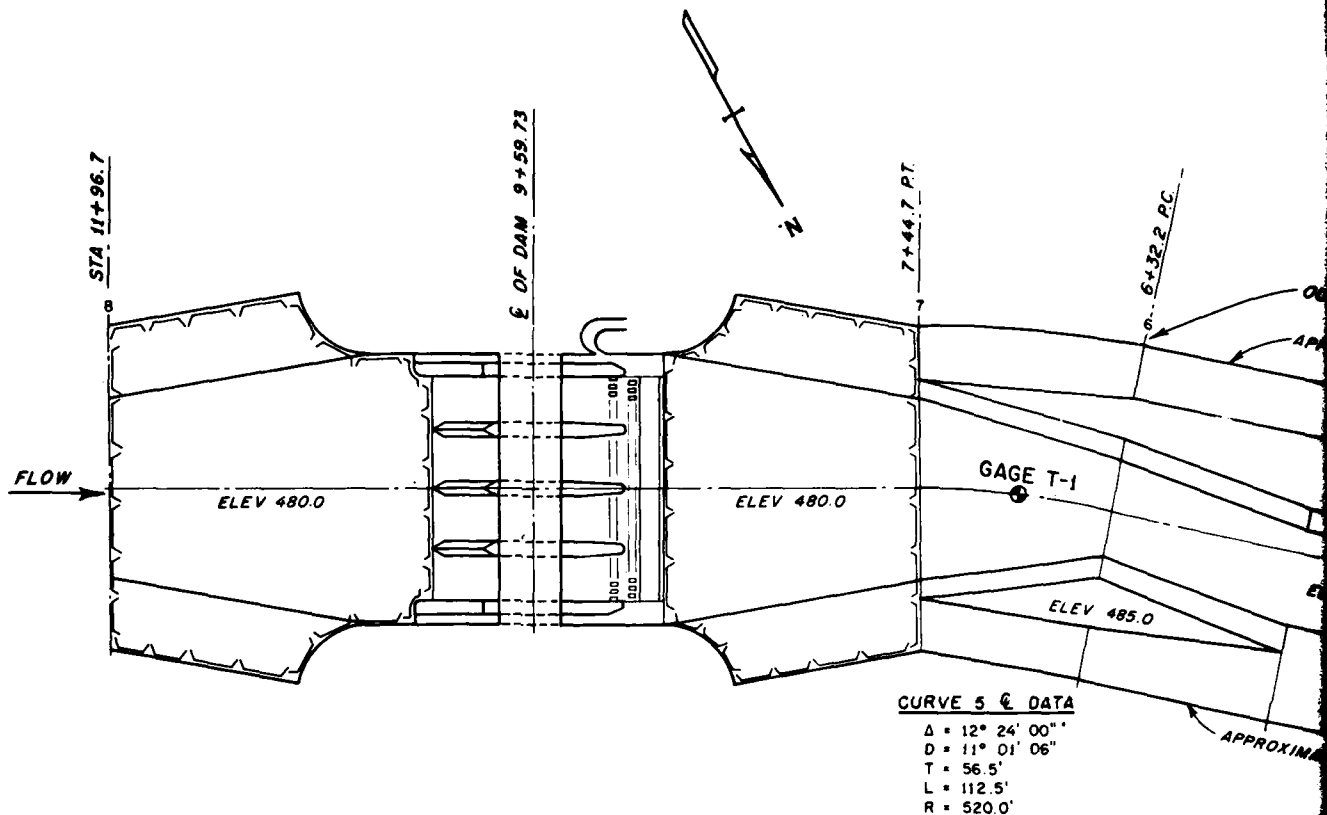
Δ = 72° 00' 00"
 D = 7° 57' 28"
 T = 523.1
 L = 904.8
 R = 720.0



SECTION	STATION
8	11+96.7
9	12+59.7
10	13+59.7
11	14+80.0
12	15+80.0
13	16+80.0
14	17+80.0
15	18+80.0
16	19+80.0
17	20+80.0
18	21+80.0
19	22+80.0
20	23+80.0
21	25+00.0
22	26+10.0
23	27+20.0
24	28+30.0
25	29+40.0

OUTLET CHANNEL DETAILS PLAN B

STATION 11+96.7 TO 29+40.0



7+44.7 P.T.

6+32.2 P.C.

3+22.2 P.T.

STA 0+00 OUTLET CHANNEL P.C.
RIVER STA 72+25.0

CURVE 6 & DATA

$\Delta = 28^\circ 24' 00''$
 $D = 8^\circ 48' 53''$
 $T = 164.5'$
 $L = 322.2'$
 $R = 650.0'$

OUTLET CHANNEL SECTIONS
 APPROXIMATE ELEV 500

GAGE T-1

ELEV 485.0

ELEV 485.0

ELEV 479.0

GAGE T-2

GAGE T-3

ELEV 478.0

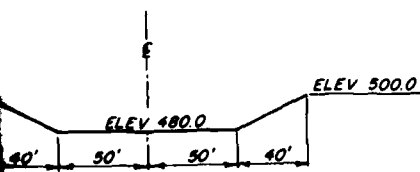
GAGE 5

CURVE 5 & DATA

$\Delta = 12^\circ 24' 00''$
 $D = 11^\circ 01' 06''$
 $T = 56.5'$
 $L = 112.5'$
 $R = 520.0'$

APPROXIMATE ELEV 500

SECTION	STATION
1	1+00.0
2	2+00.0
3	3+22.2
4	4+22.2
5	5+22.2
6	6+32.2
7	7+44.7
8	11+96.7

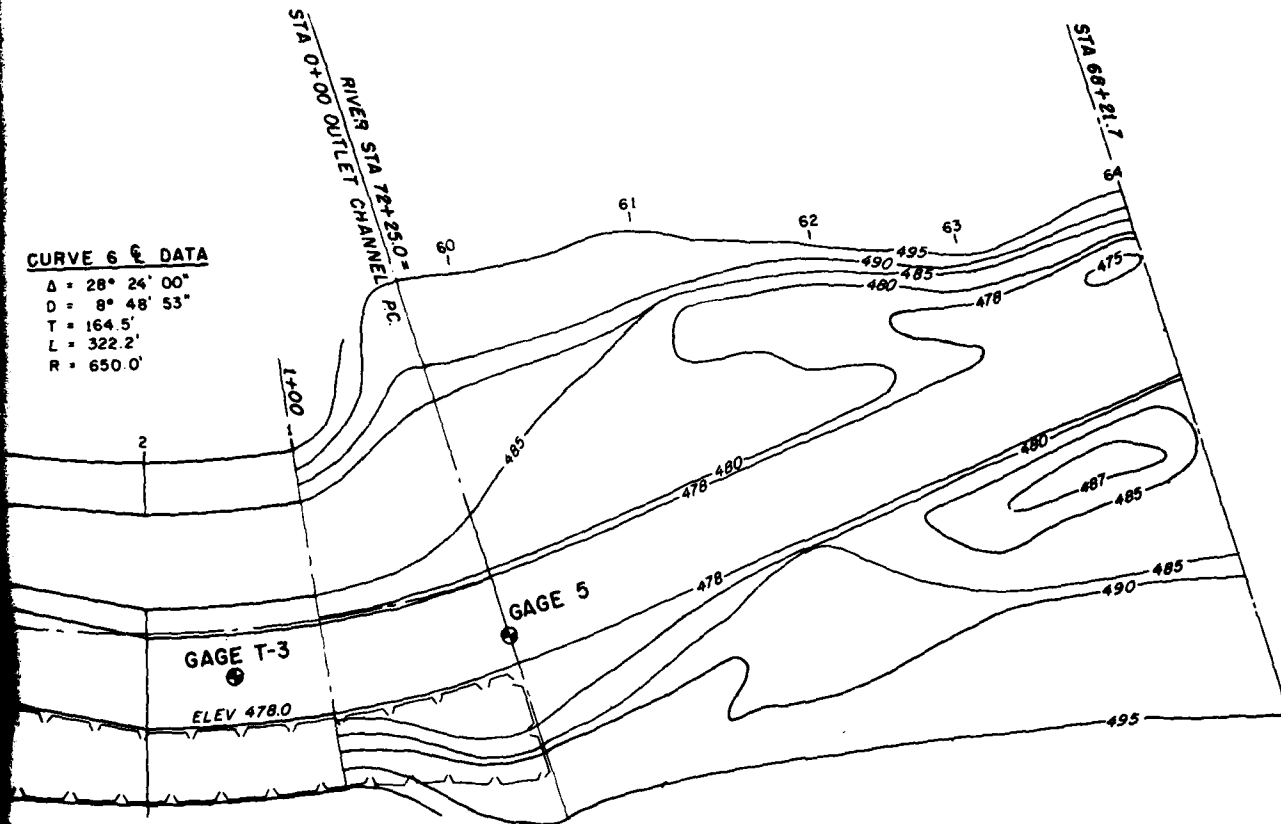


SECTIONS 7 AND 8

2

CURVE 6 & DATA

Δ = 28° 24' 00"
 D = 8° 48' 53"
 T = 164.5'
 L = 322.2'
 R = 650.0'



STATION

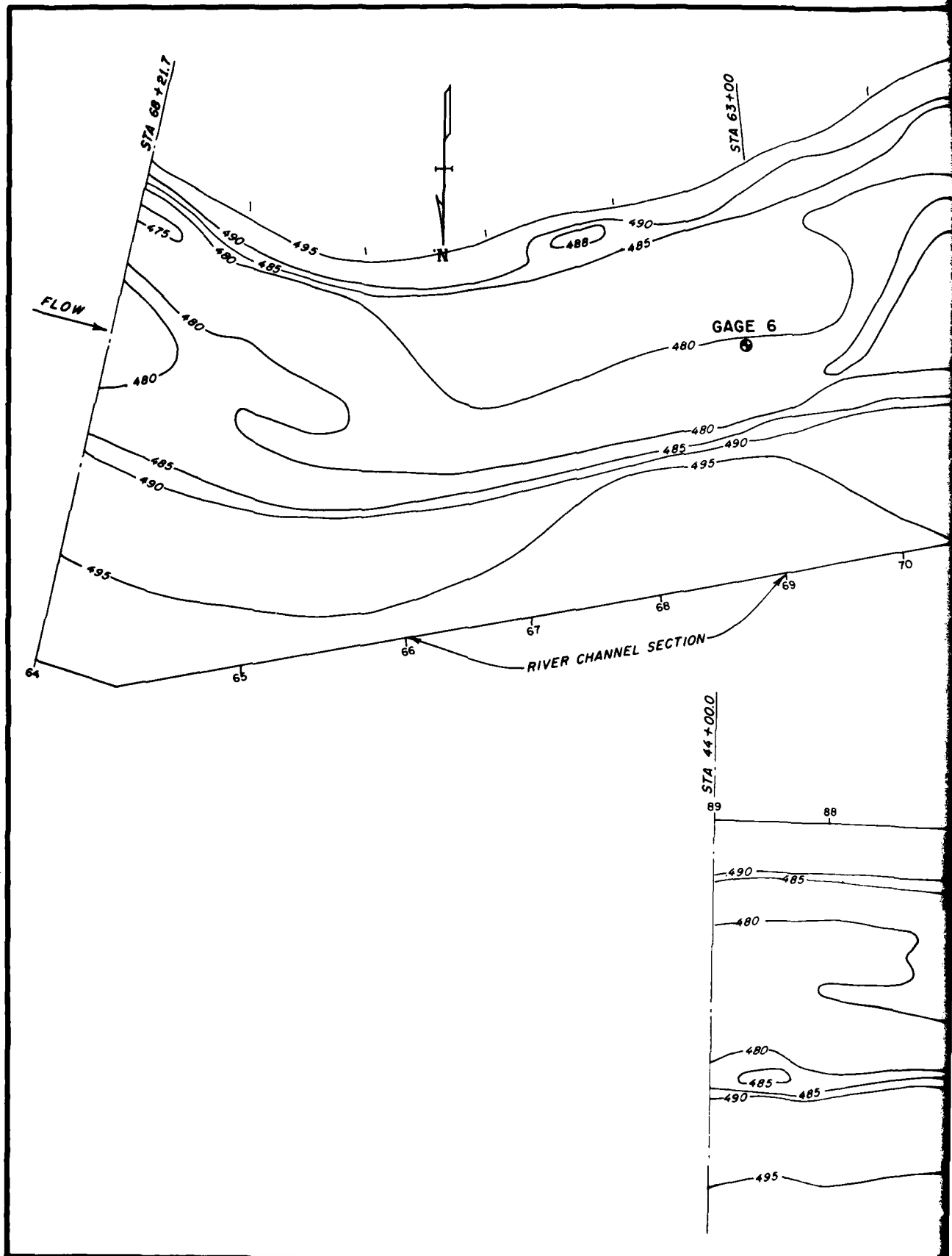
1+00.0
 2+00.0
 3+22.2
 4+22.2
 5+22.2
 6+32.2
 7+44.7
 11+96.7

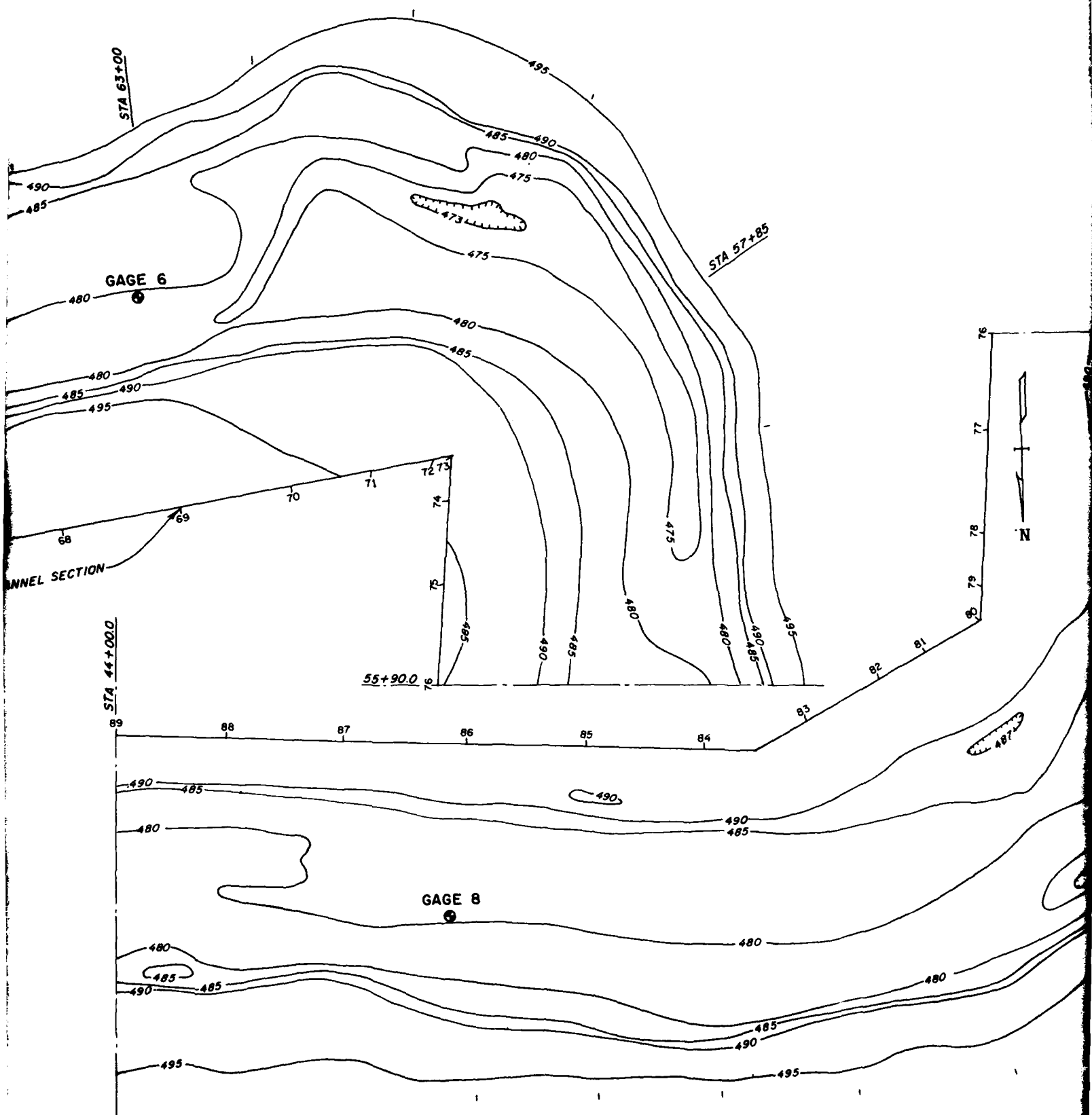
**OUTLET CHANNEL DETAILS
 PLAN B
 STATION 0+00 TO 11+96.7**

2

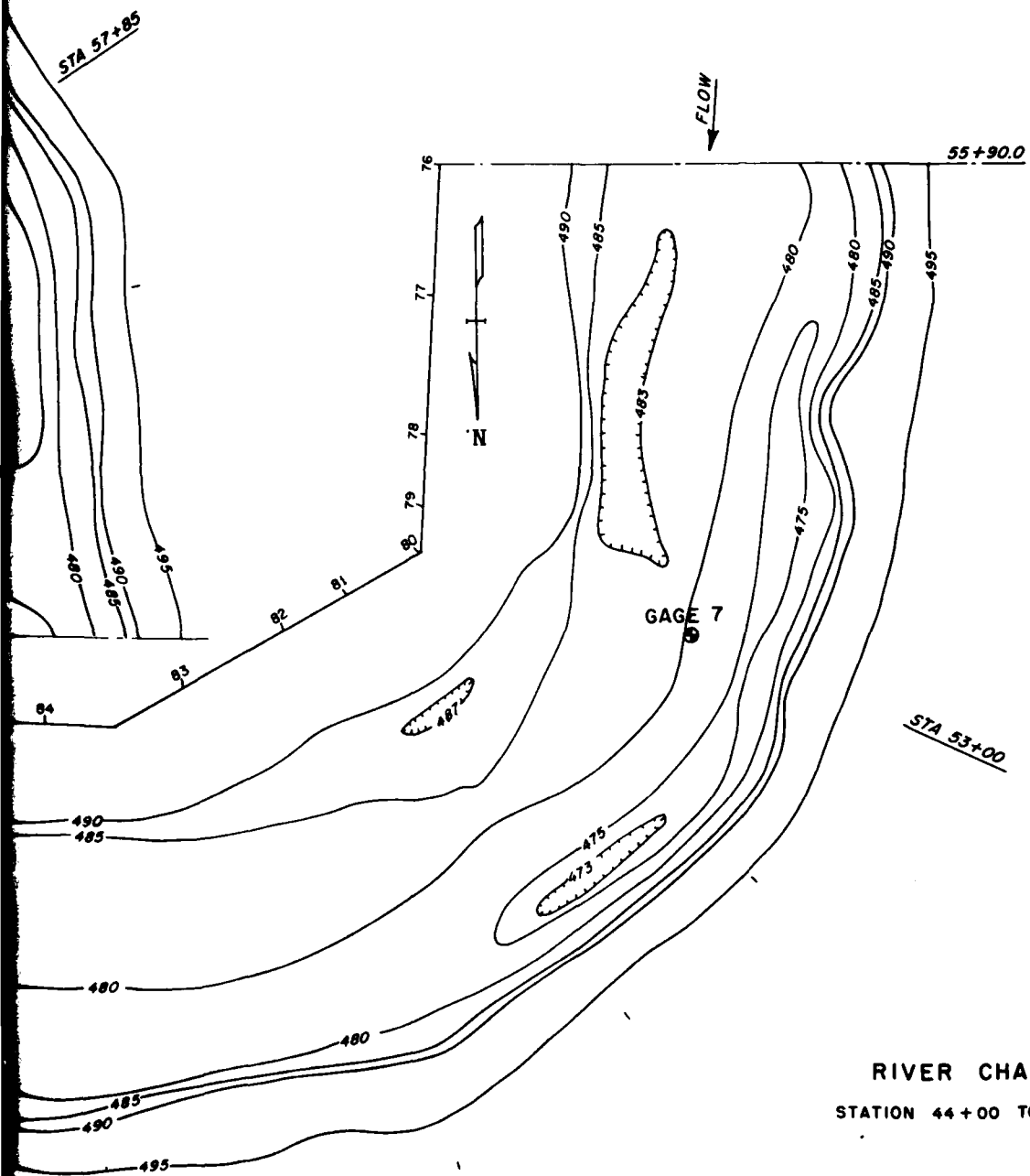
1

3





2

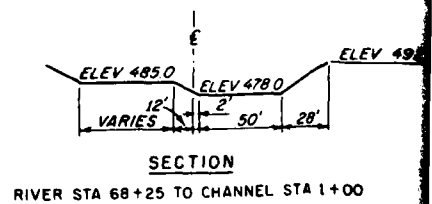
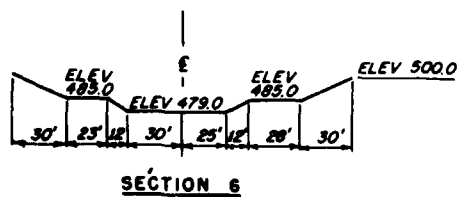
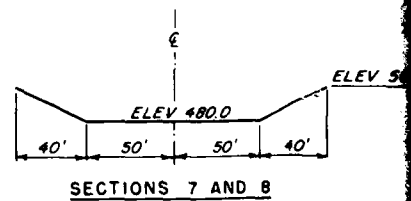
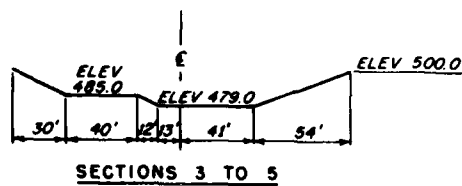
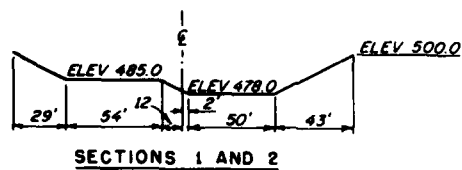
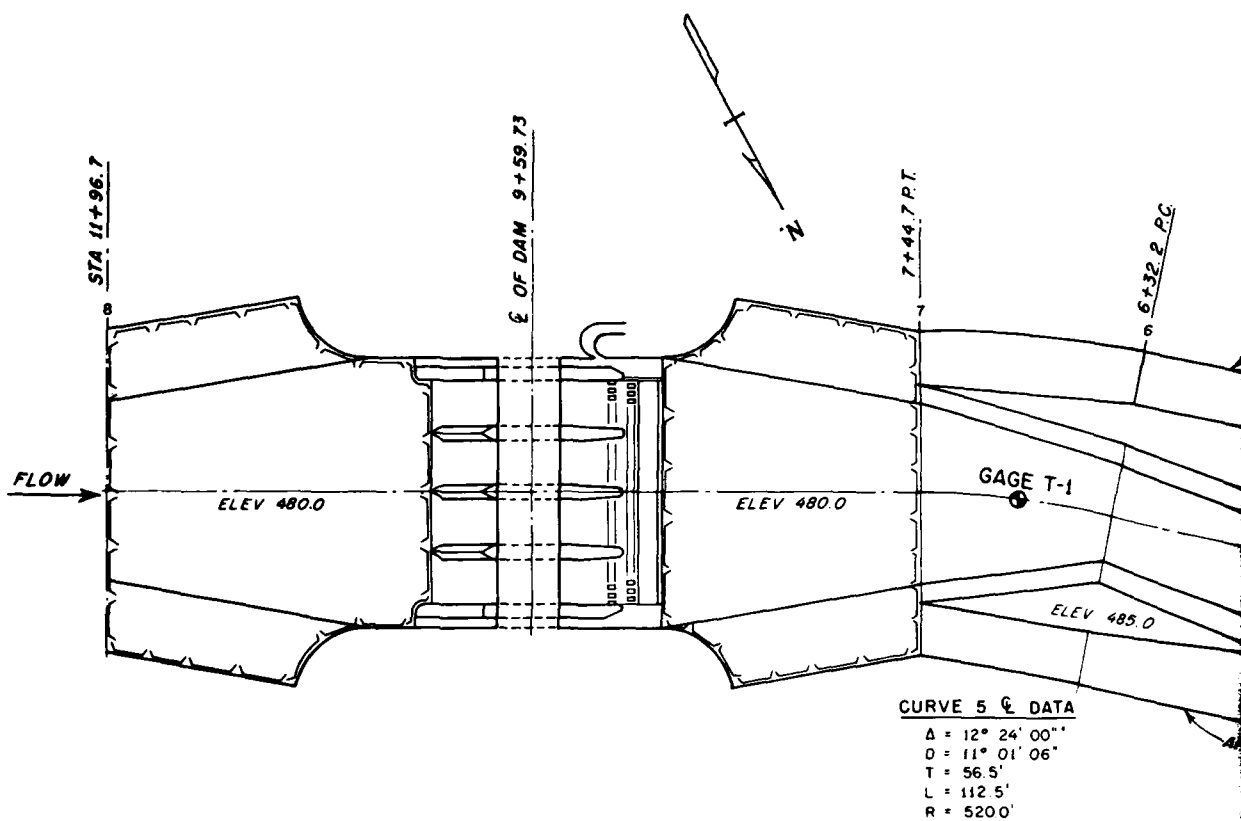


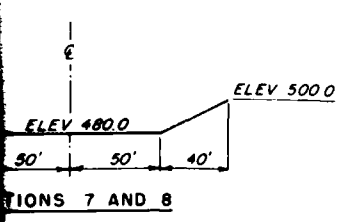
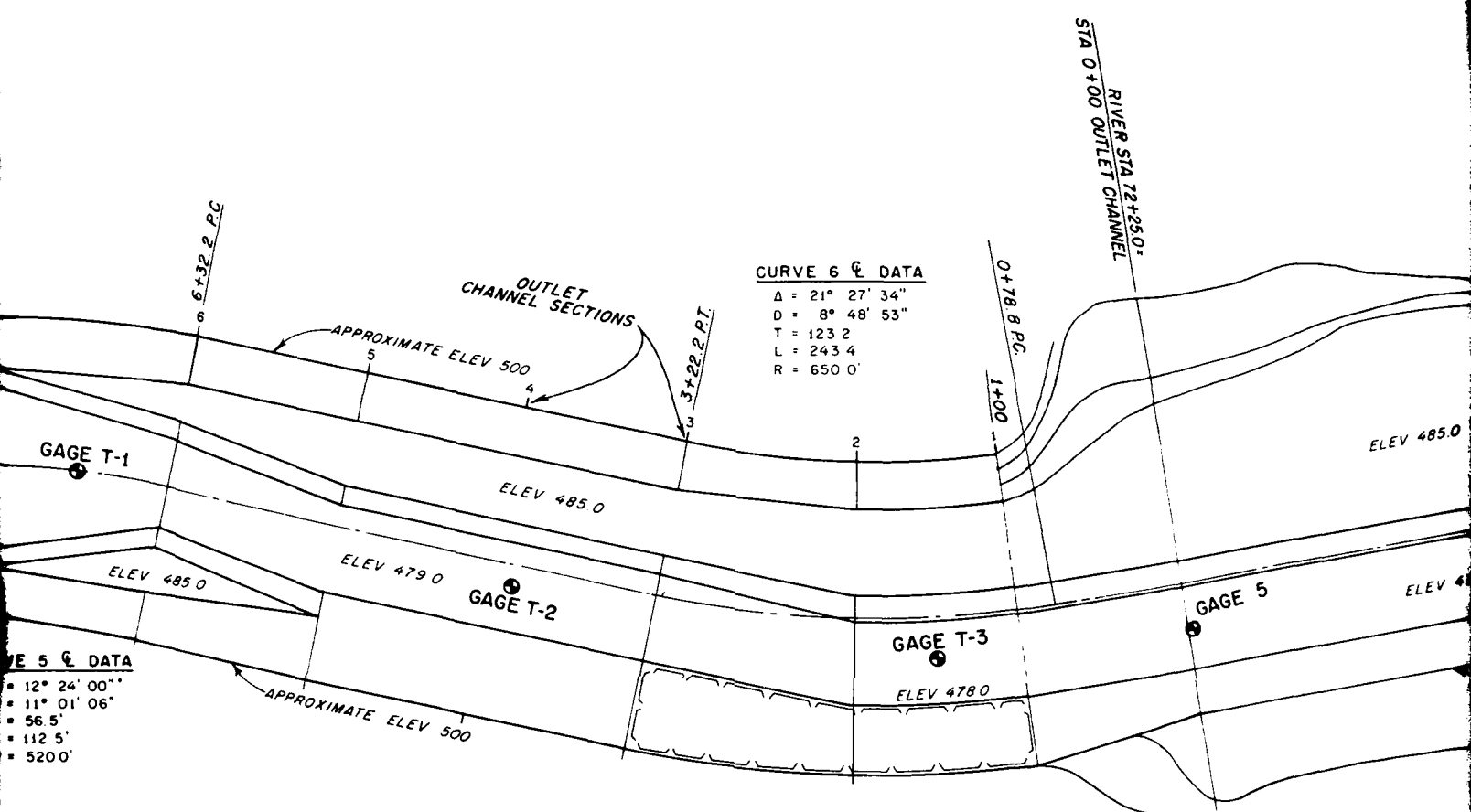
RIVER CHANNEL
STATION 44+00 TO 68+21.7

2

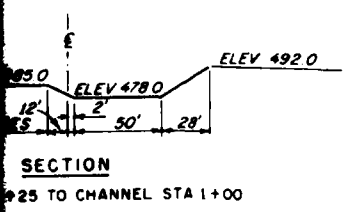
1

3

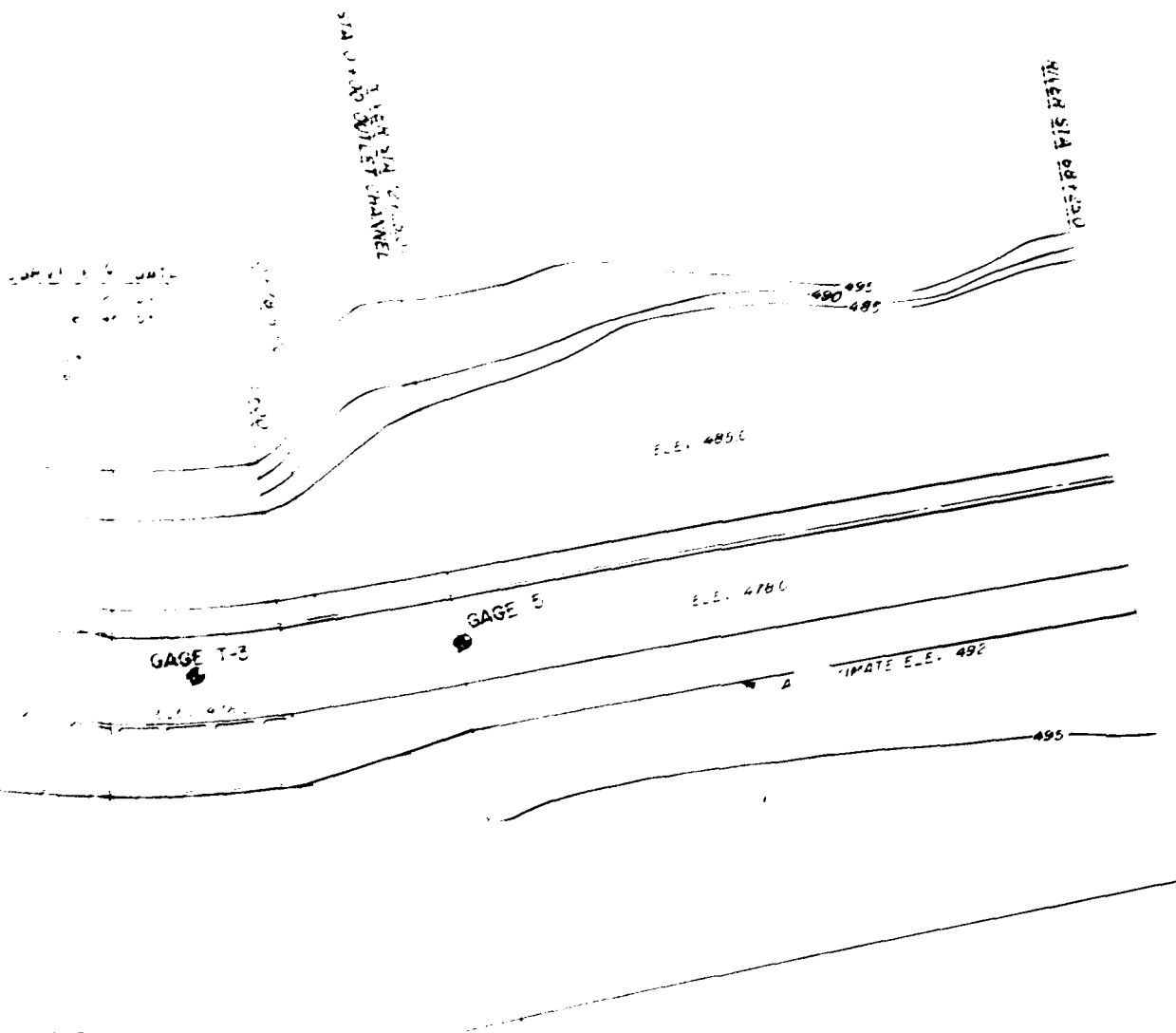




SECTION	STATION
1	1+00.0
2	2+00.0
3	3+22.2
4	4+22.2
5	5+22.2
6	6+32.2
7	7+44.7
8	11+96.7



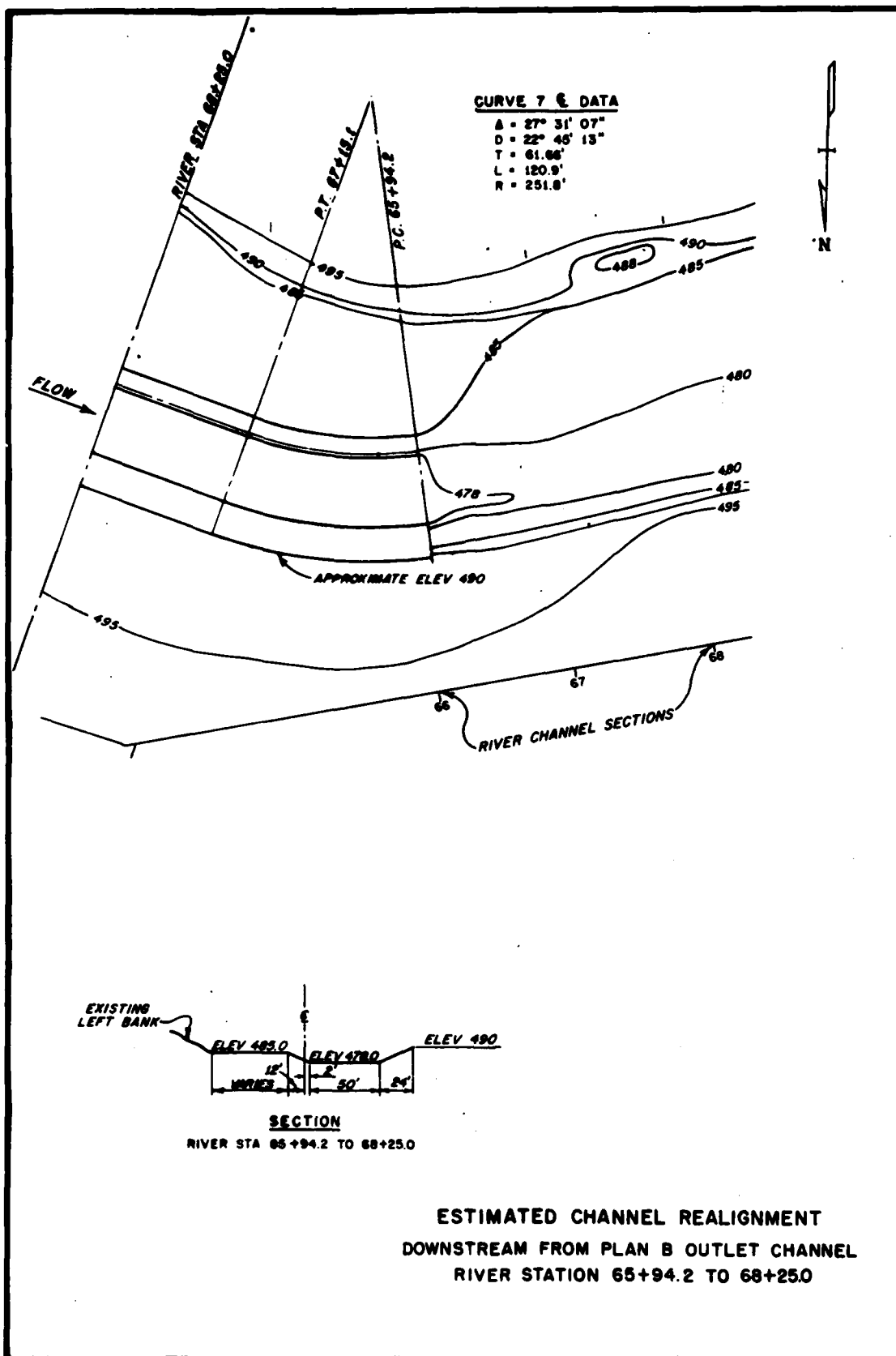
ESTIMATE

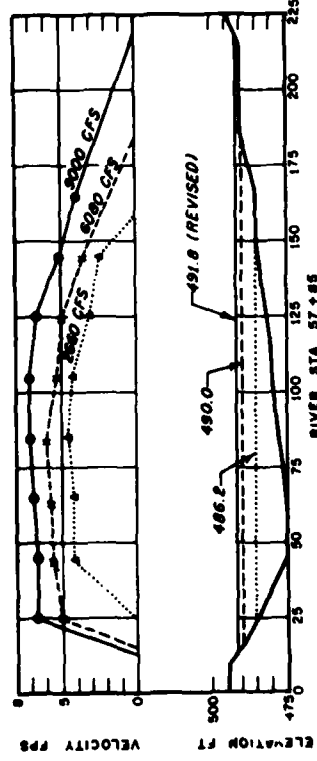
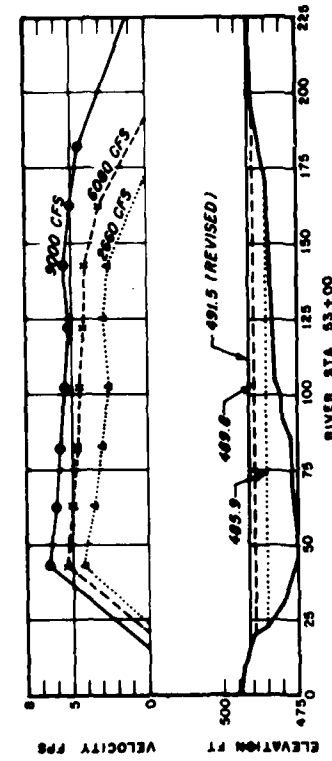
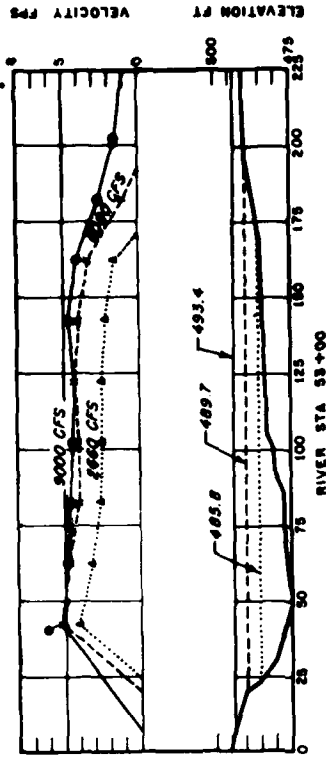
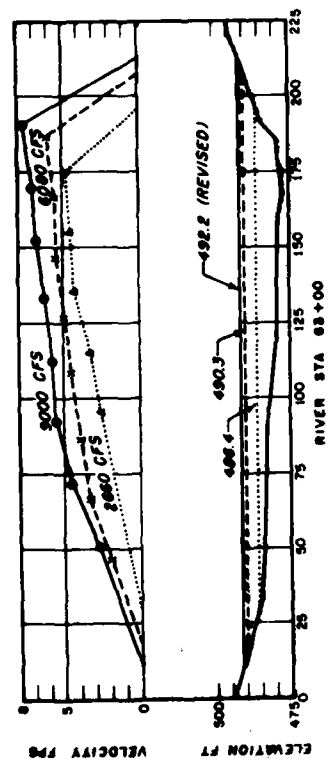
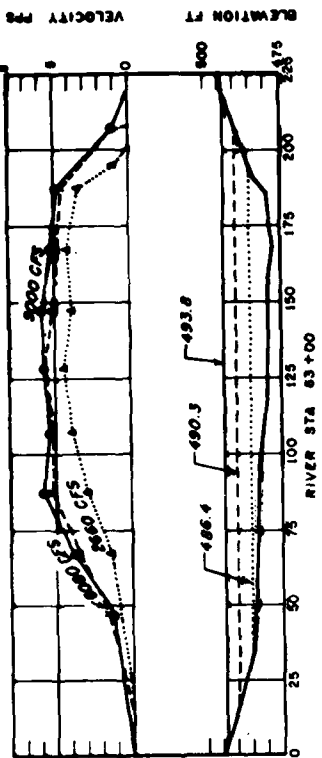


STATION
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00

PLAN C OUTLET STRUCTURE

PLAN B OUTLET CHANNEL
AND
ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
CHANNEL STATION 0+00 TO 11+96.7
RIVER STATION 68+25.0 TO 72+25.0



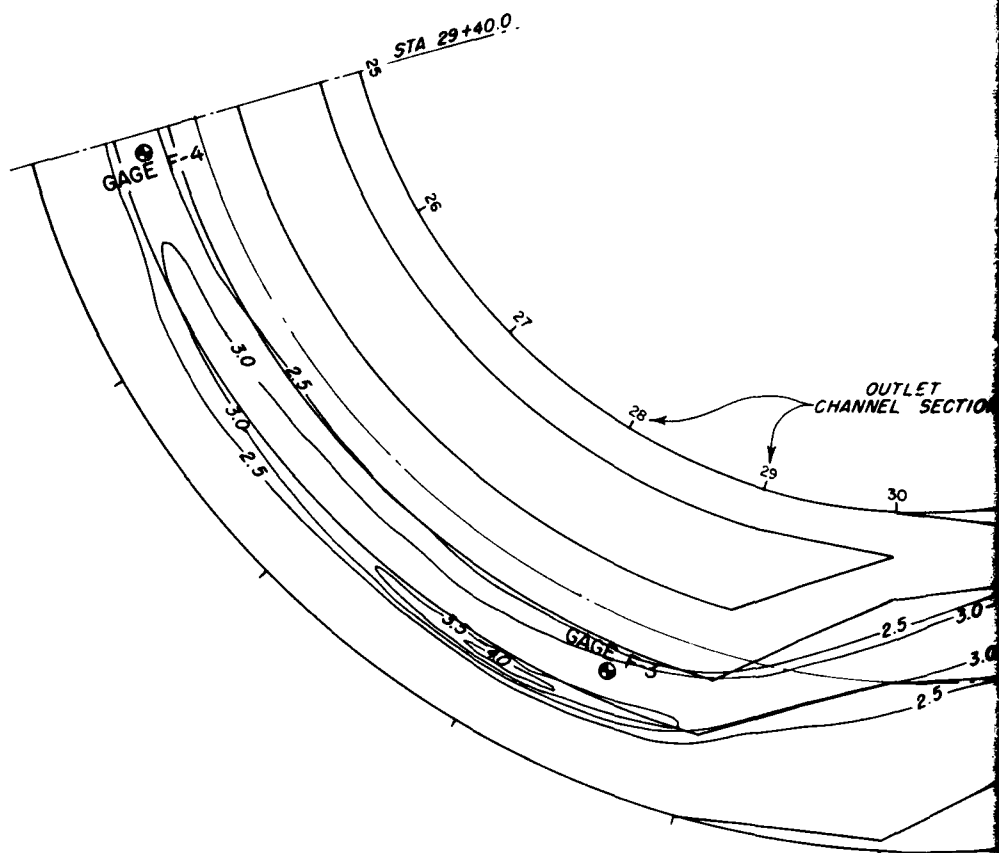


NOTES

1. VELOCITIES IN FPS AT Q8-DEPTH.
2. AVERAGE WATER SURFACE ELEVATIONS SHOWN.
3. SECTIONS LOOKING DOWNSTREAM.

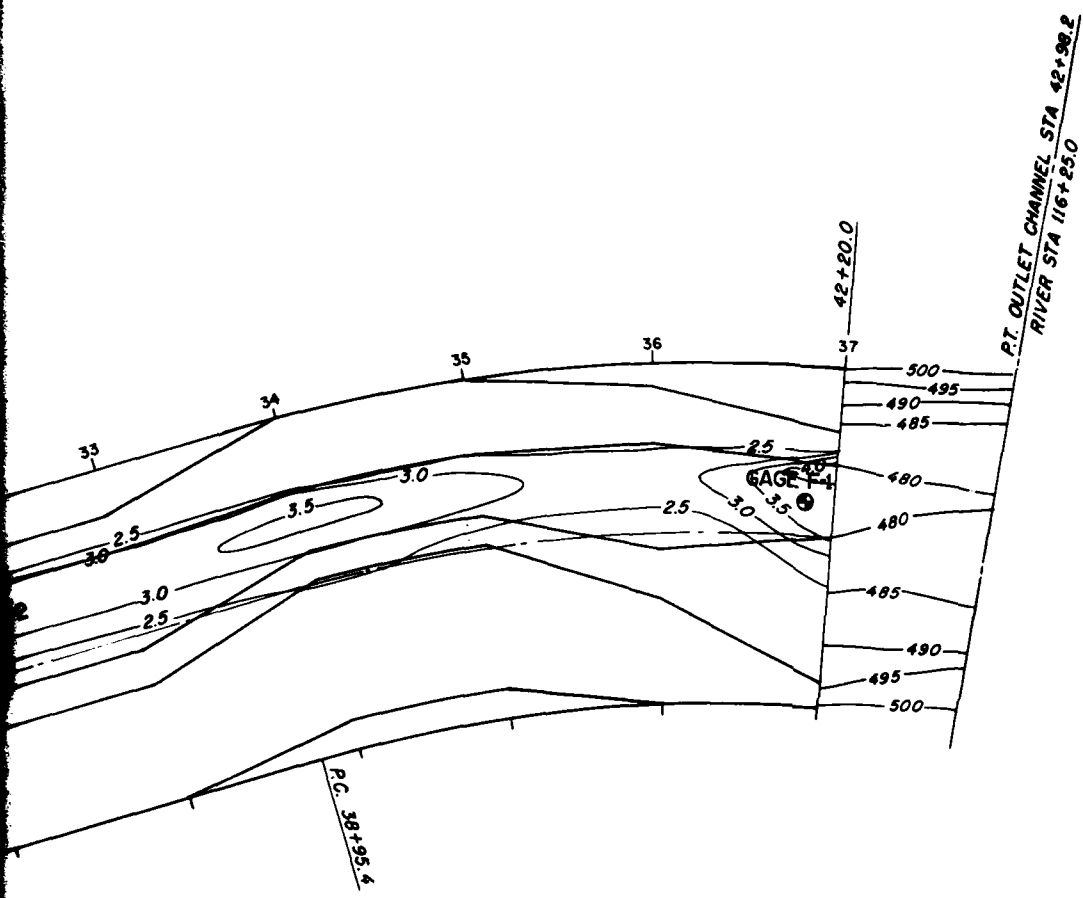
**WATER SURFACES AND VELOCITIES
PLAN B CHANNEL
WITH
ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
RIVER STATIONS 53+00, 57+85, AND 63+00**

PLAN B OUTLET CHANNEL



NO
1 PLAN C OUTL
2. VELOCITIES
ABOVE BOTTO

2

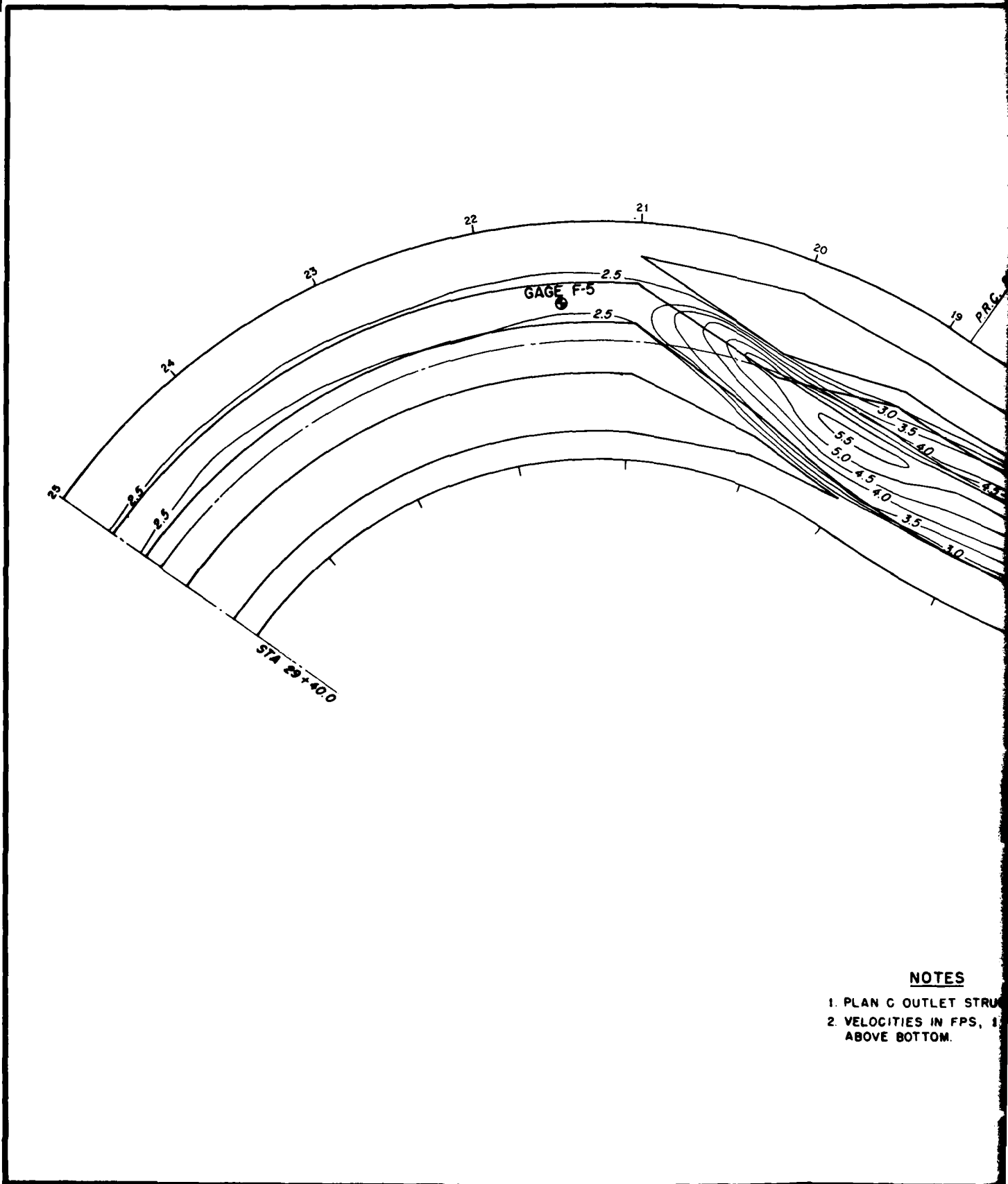


GAGE	WATER-SURFACE ELEVATIONS
F - 1	487.9
F - 2	487.7
F - 3	487.5
F - 4	487.4

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 29+40.0 TO 42+20.0
 DISCHARGE 1630 CFS
 UNGATED FLOW

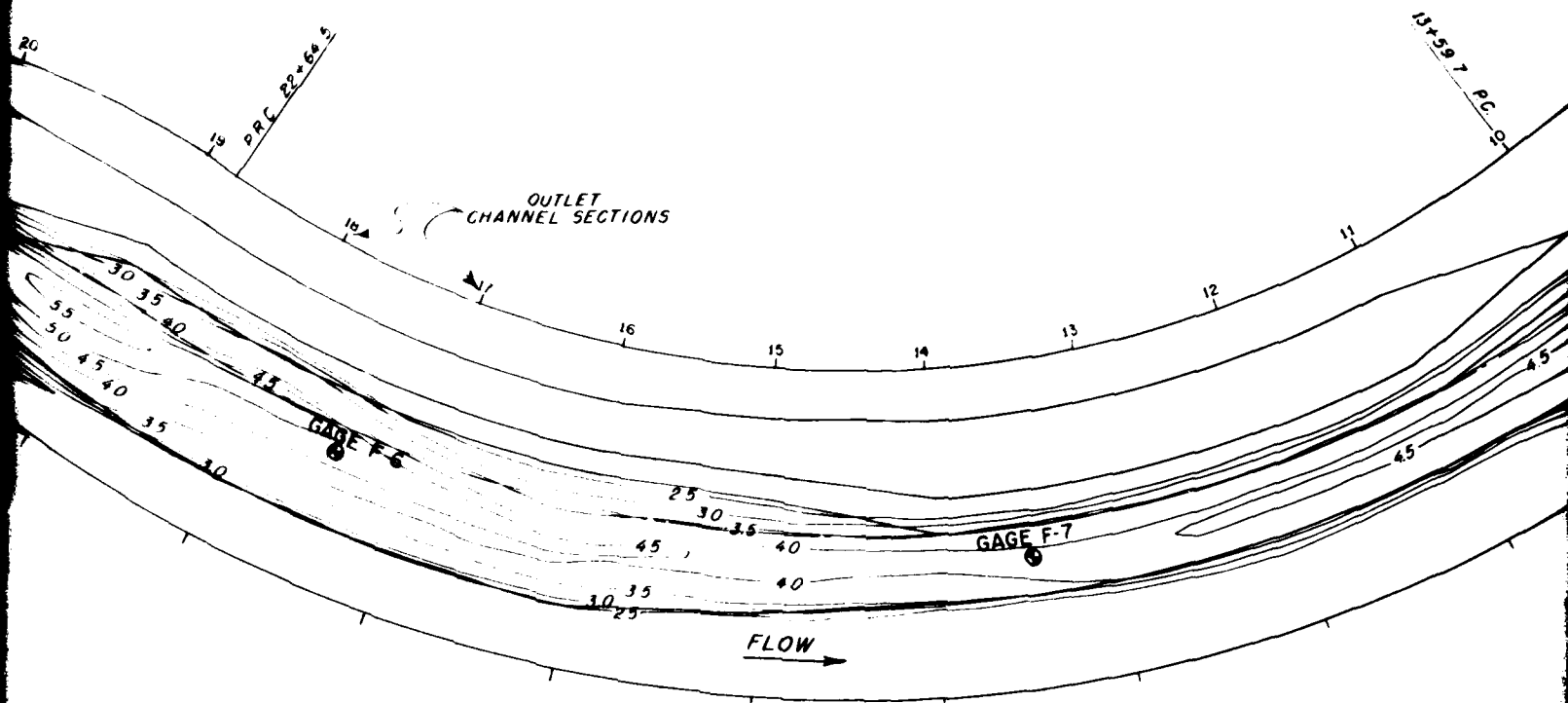
2

3



NOTES

1. PLAN C OUTLET STRU
2. VELOCITIES IN FPS, 1
ABOVE BOTTOM.

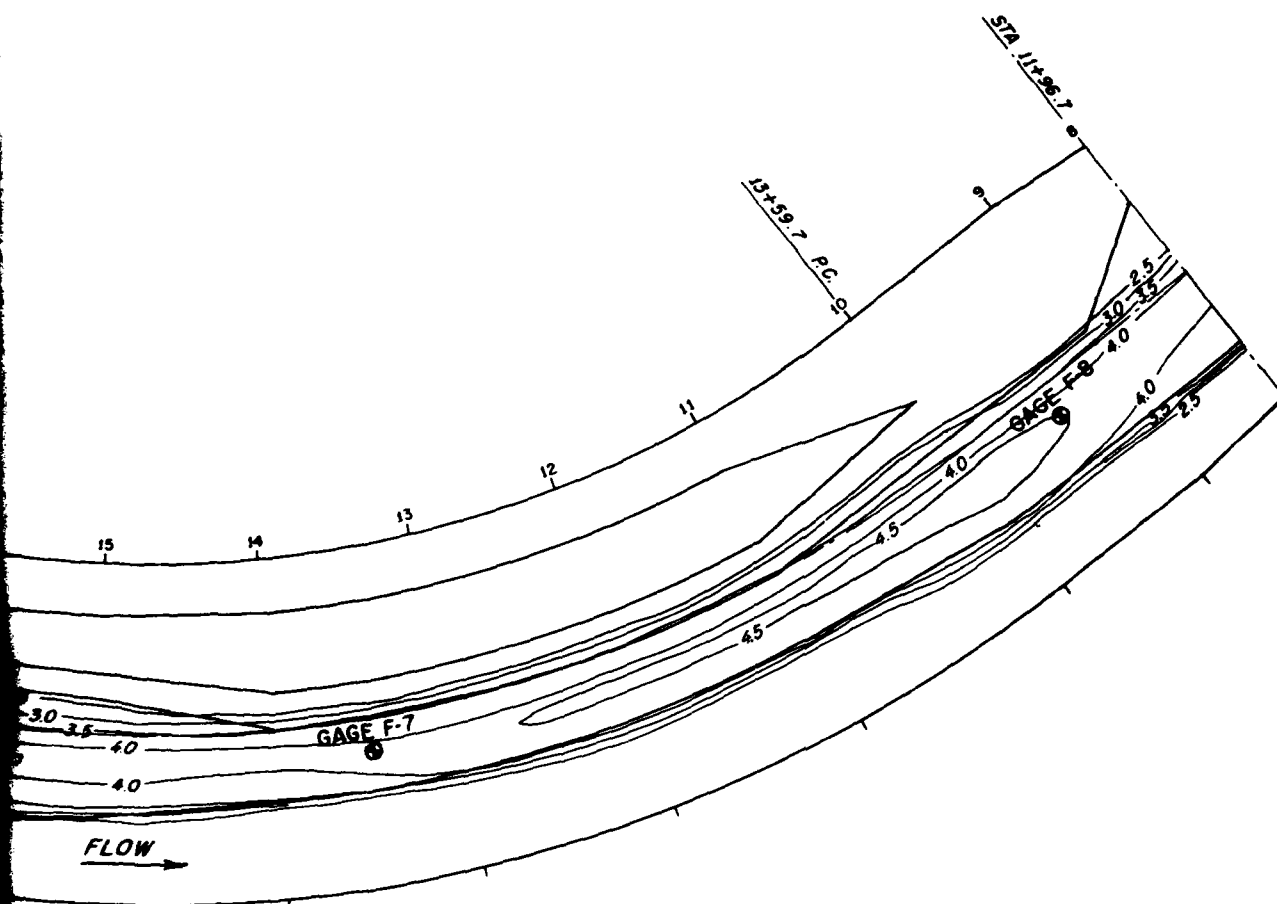


NOTES

- 1 PLAN C OUTLET STRUCTURE
- 2 VELOCITIES IN FPS, 1 TO 15 FT ABOVE BOTTOM

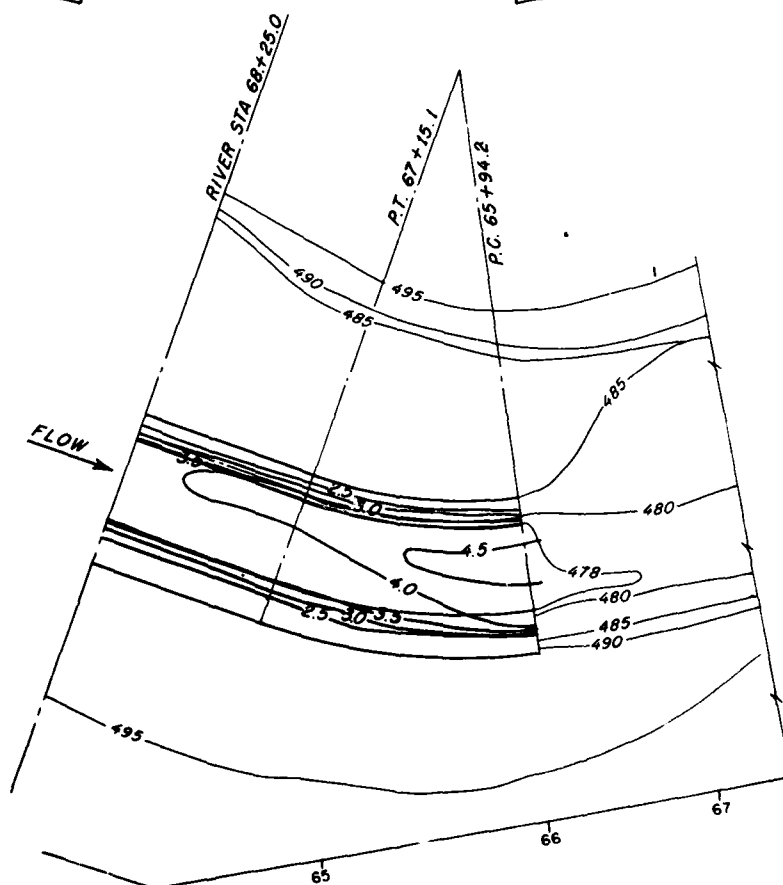
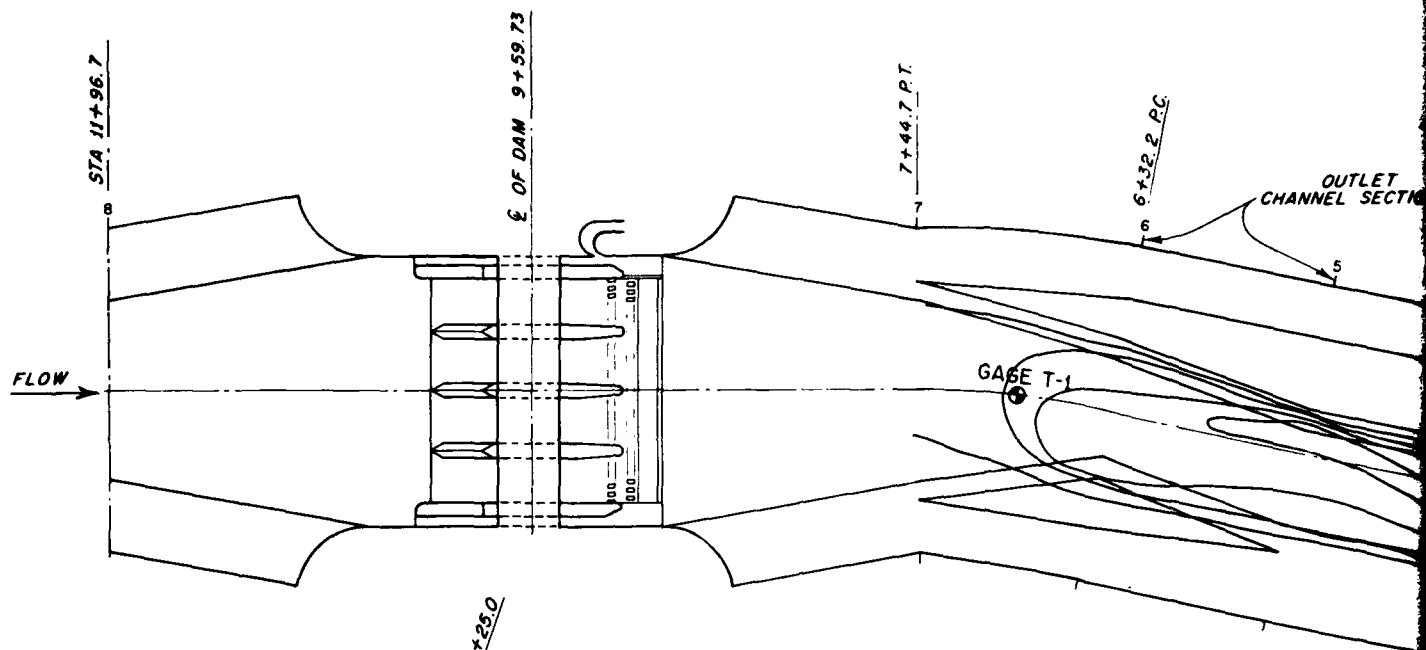
GAGE	WATER-SURFACE ELEVATIONS
F-5	487.2
F-6	486.8
F-7	486.4
F-8	486.1

BOTT
PLAN
STATION
DISC
U



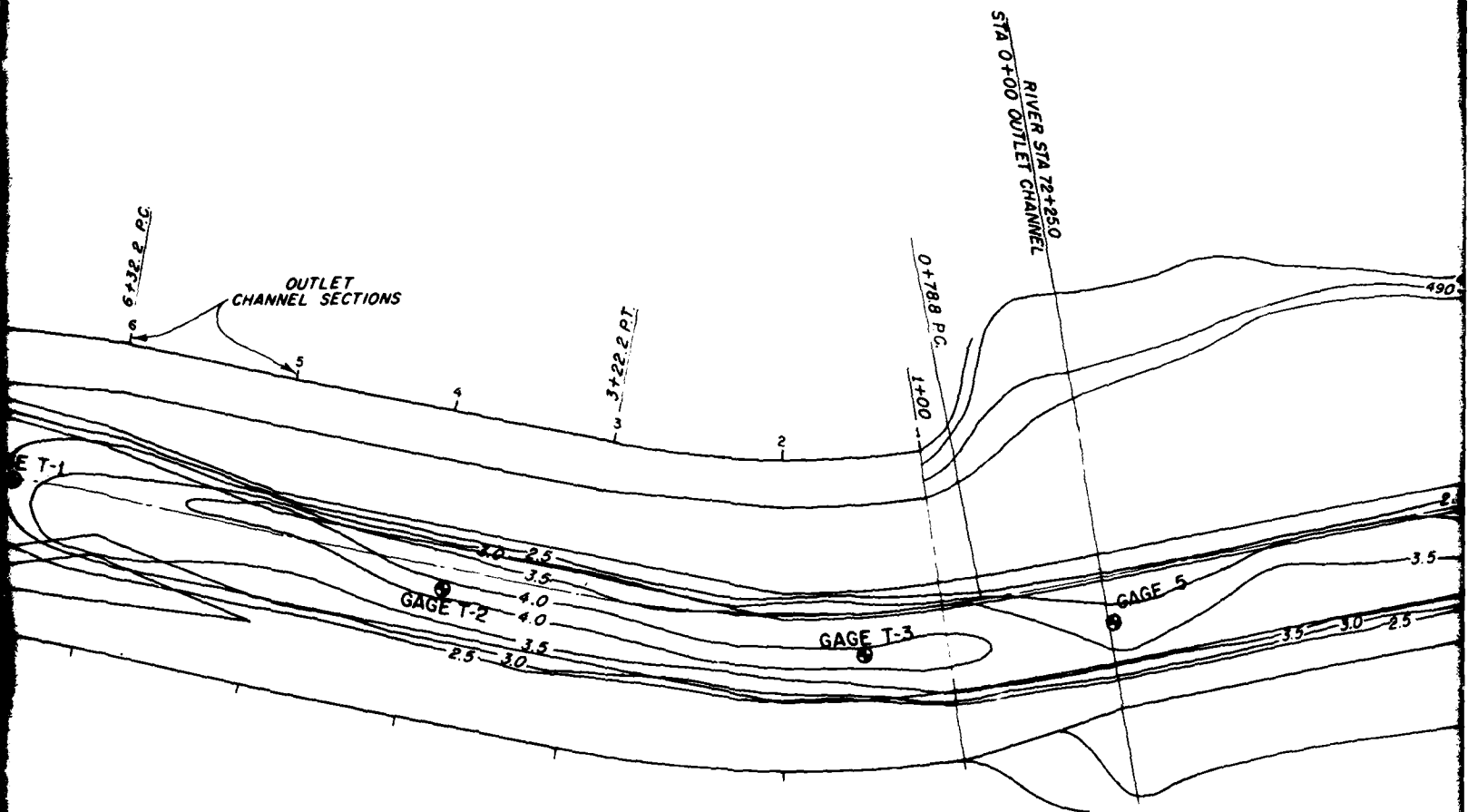
GAGE	WATER-SURFACE ELEVATIONS
F-5	487.2
F-6	486.8
F-7	486.4
F-8	486.1

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 11+96.7 TO 29+40.0
 DISCHARGE 1630 CFS
 UNGATED FLOW



NOTES

1. PLAN C OUTLET STRUCTURE
2. VELOCITIES IN FPS, 1' ABOVE BOTTOM.



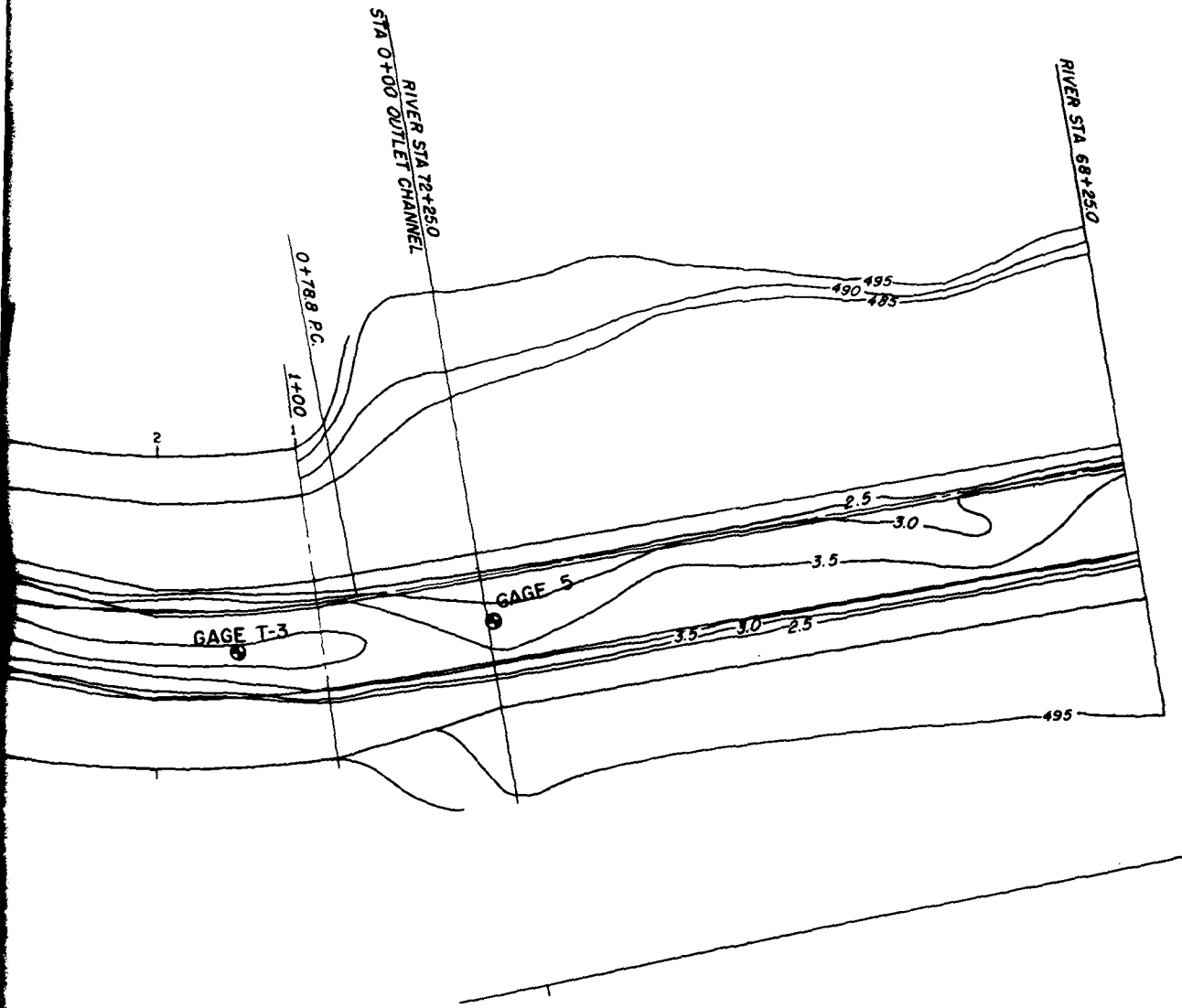
NOTES

1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.

GAGE	WATER-SURFACE ELEVATIONS
T-1	485.9
T-2	485.6
T-3	485.4
5	485.4

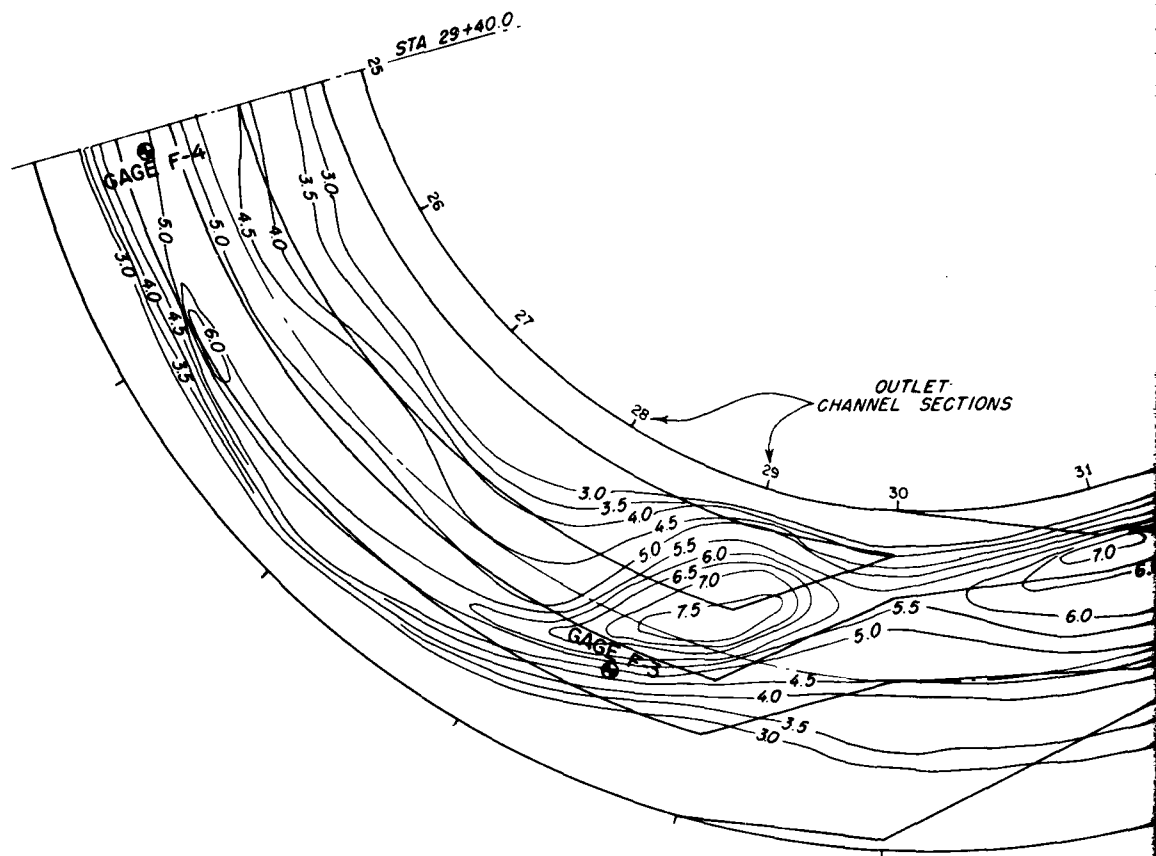
ESTIMATE
RIVER STA

2



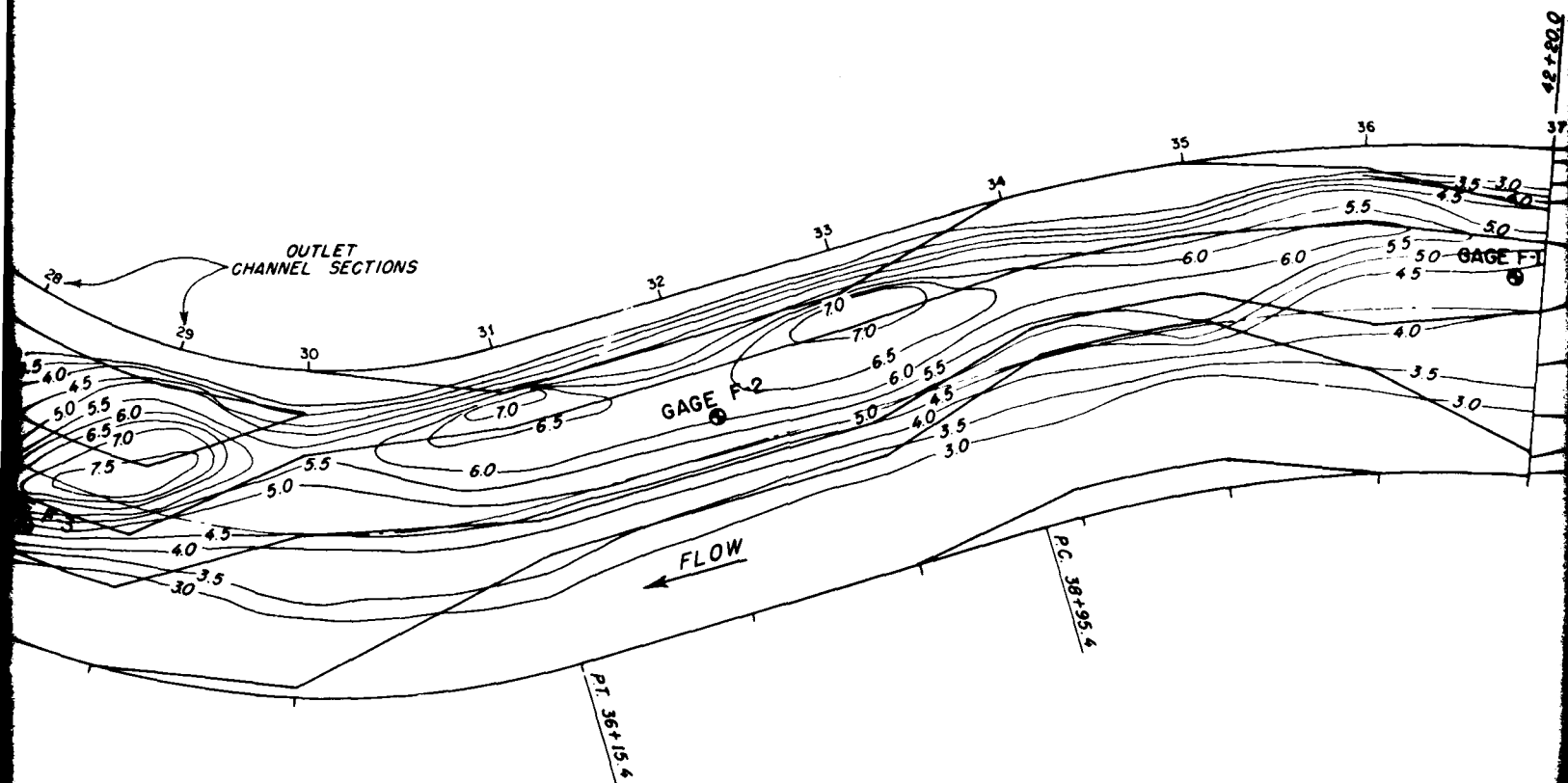
GAGE	WATER-SURFACE ELEVATIONS
T-1	485.9
T-2	485.6
T-3	485.4
5	485.4

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 AND
 ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
 RIVER STATION 65+94.2 TO CHANNEL STATION 7+44.7
 DISCHARGE 1630 CFS
 UNGATED FLOW



NOTES

1. PLAN C OUTLET STRUCTURE
2. VELOCITIES IN FPS, 1 TO 1.5 ABOVE BOTTOM.

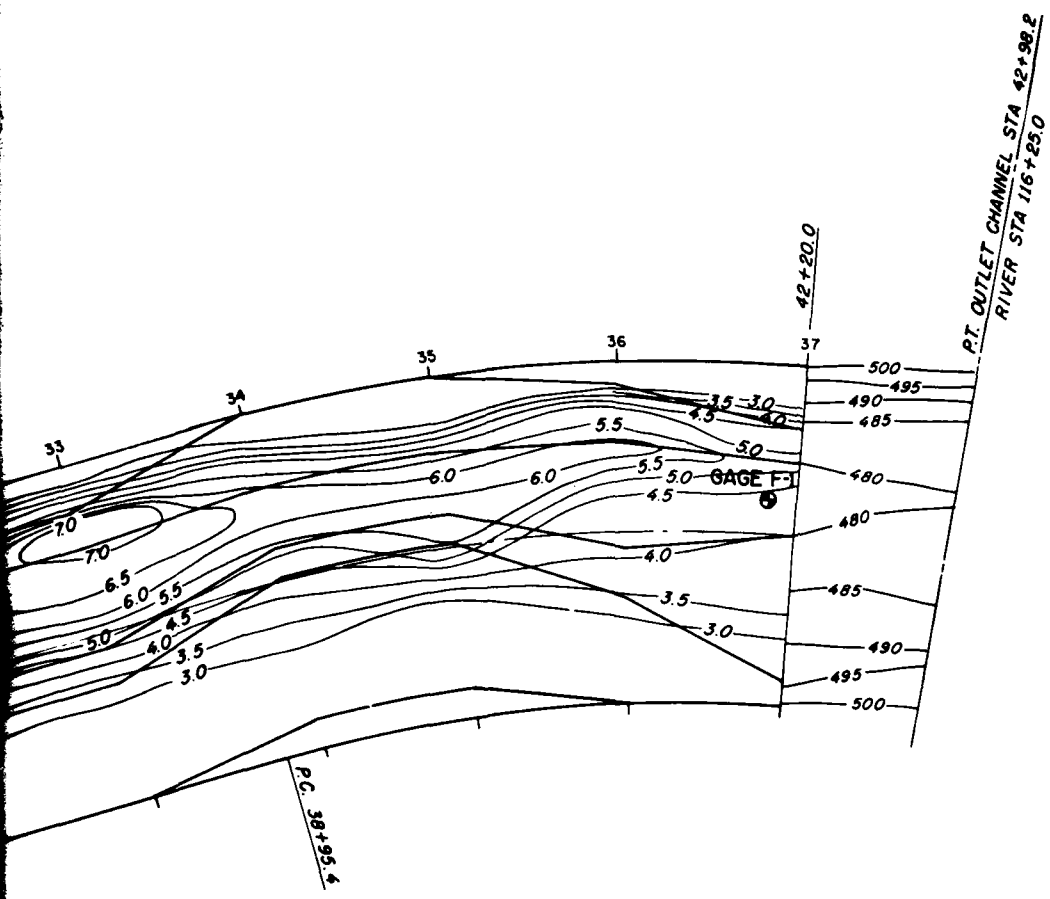


NOTES

1. PLAN C OUTLET STRUCTURE
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.

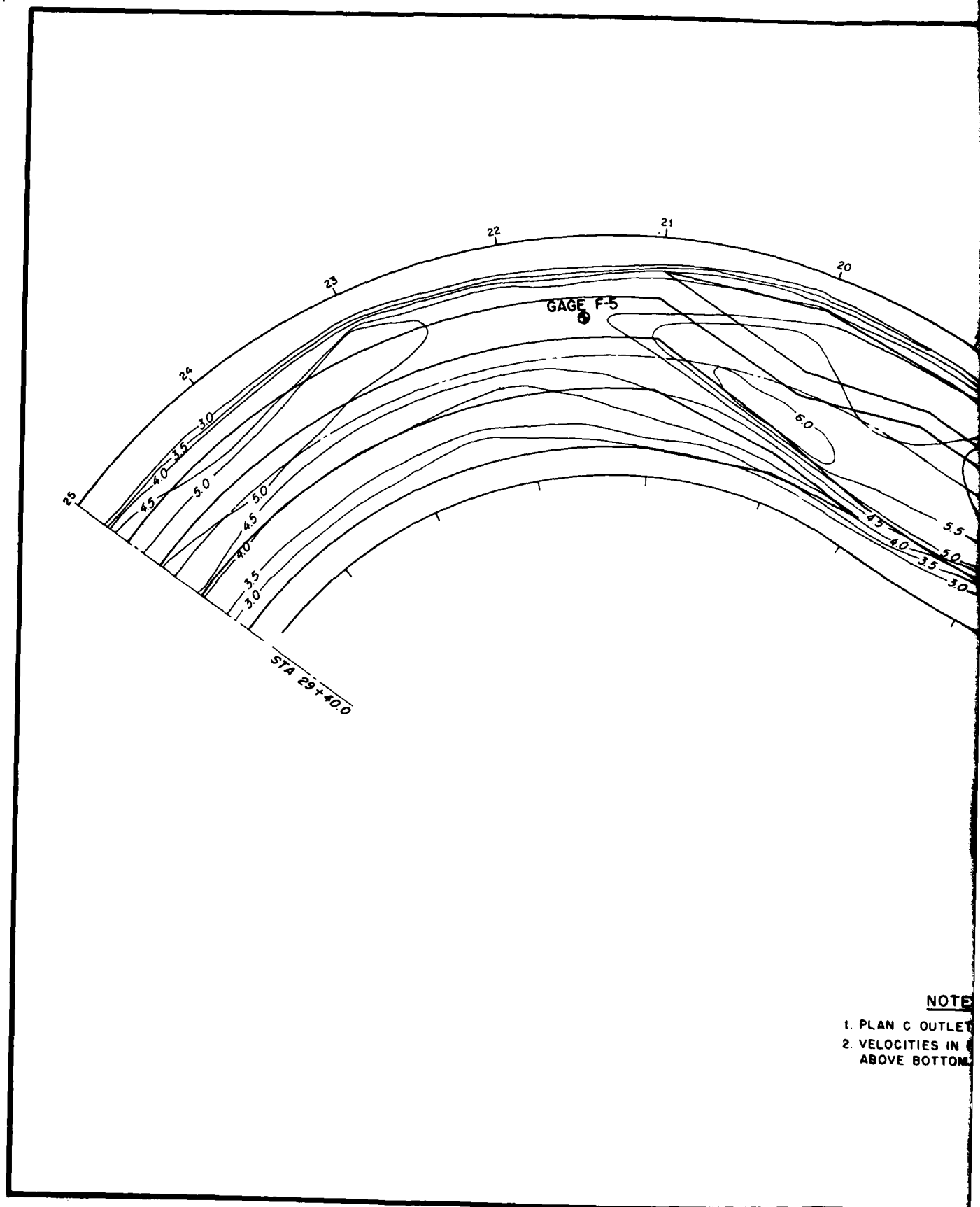
GAGE	WATER-SURFACE ELEVATIONS
F - 1	494.7
F - 2	494.2
F - 3	494.0
F - 4	493.8

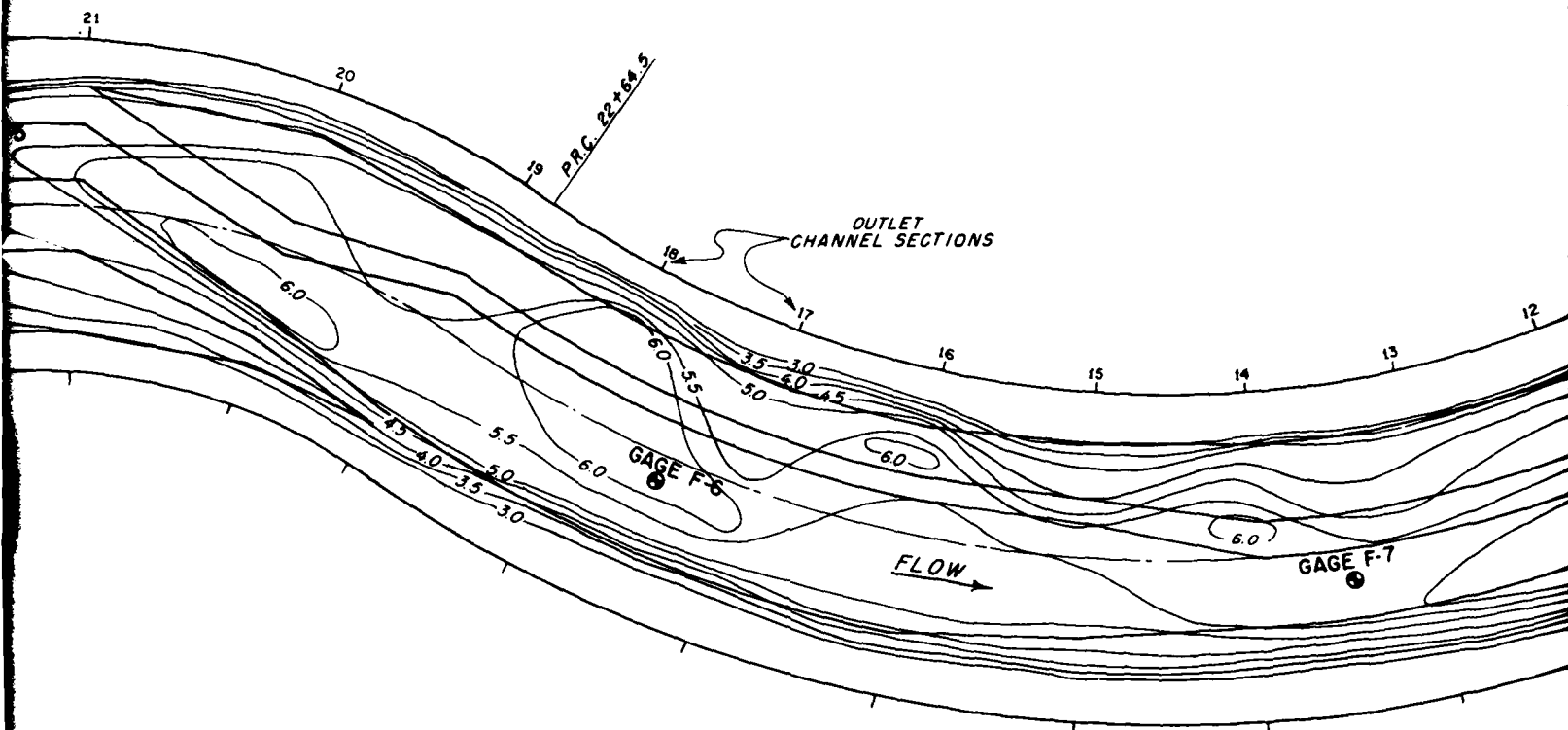
2



GAGE	WATER-SURFACE ELEVATIONS
F - 1	494.7
F - 2	494.2
F - 3	494.0
F - 4	493.8

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 29+40.0 TO 42+20.0
 DISCHARGE 7200 CFS
 UNGATED FLOW

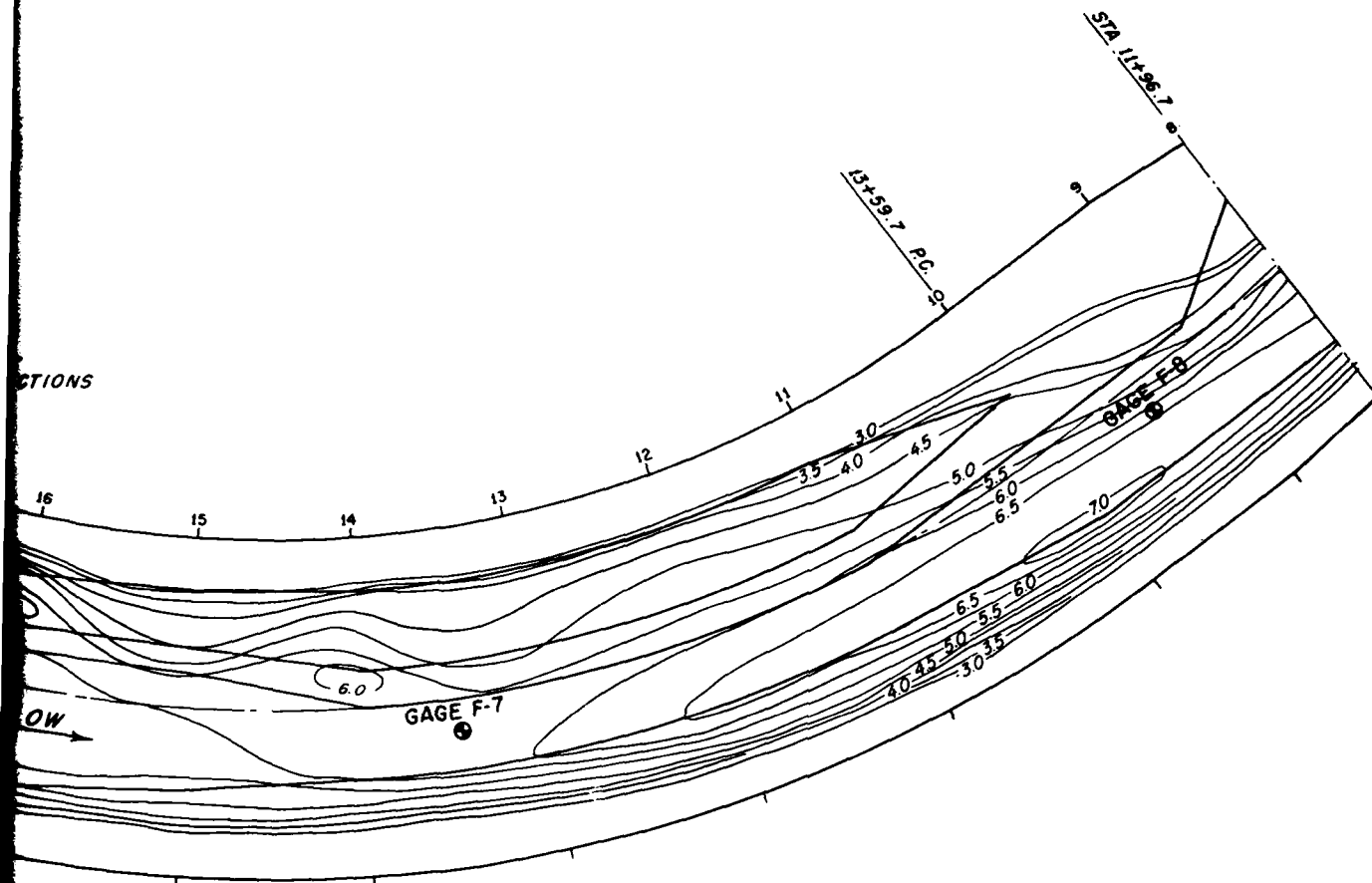




NOTES

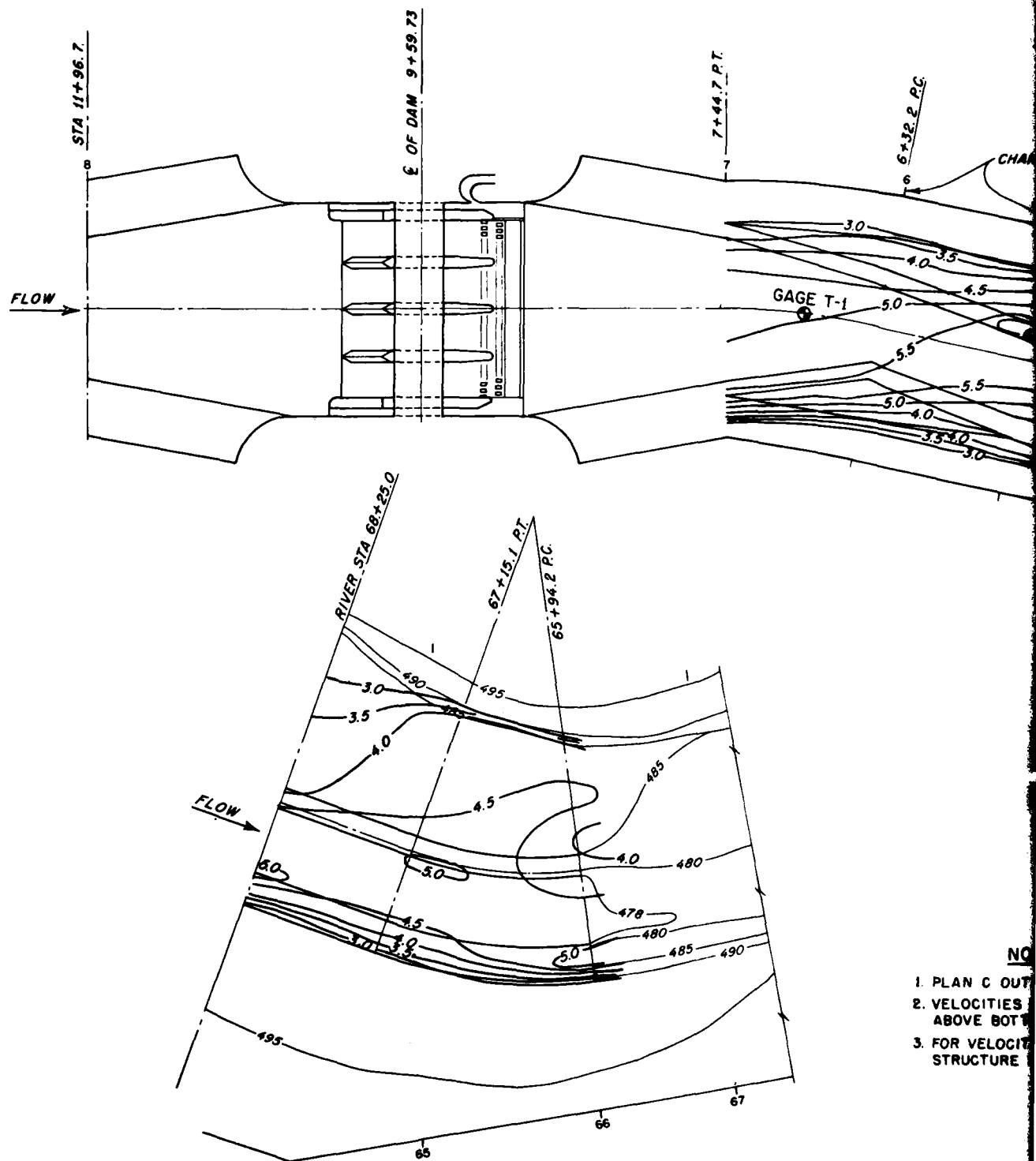
1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.

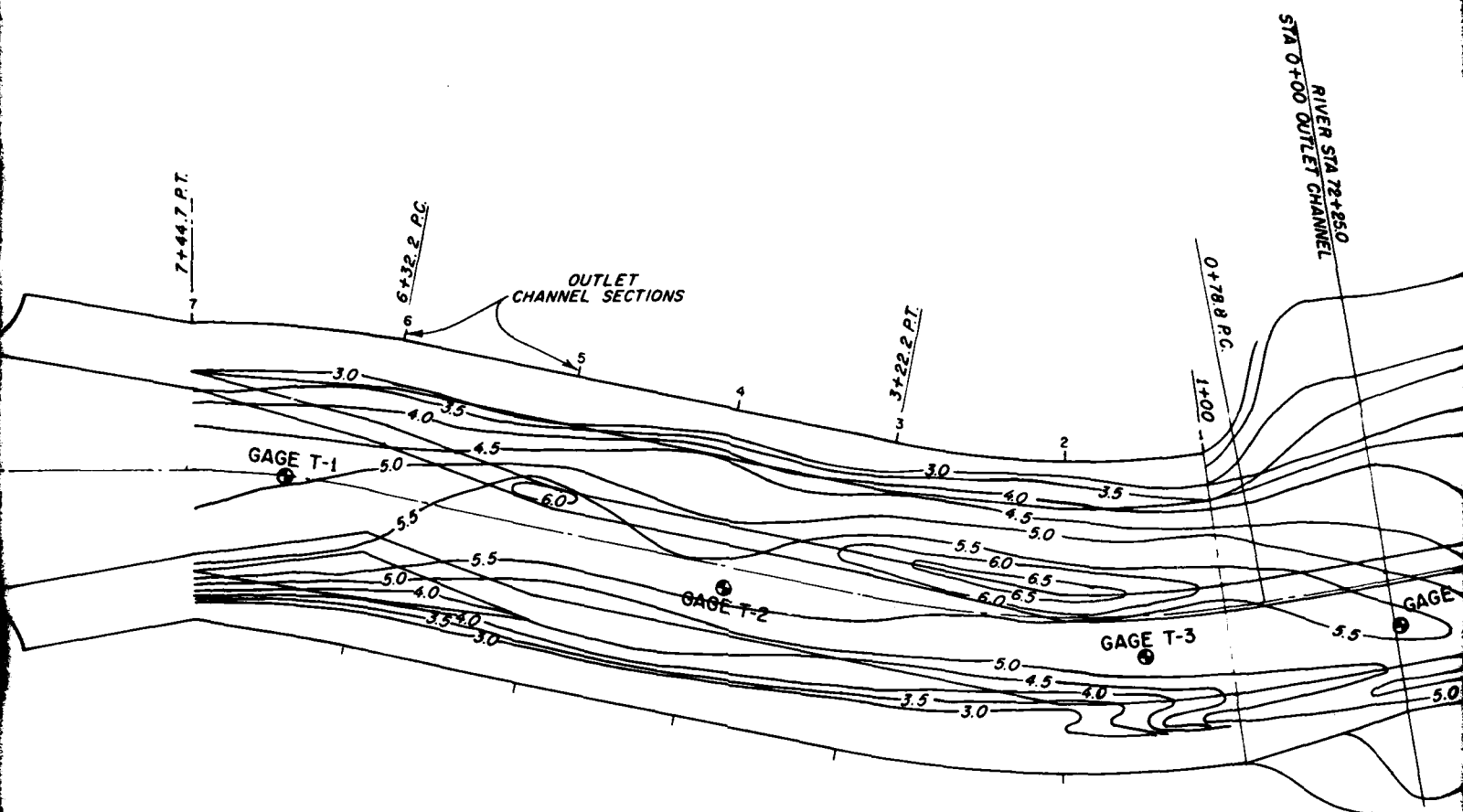
GAGE	WATER-SURFACE ELEVATIONS
F - 5	493.5
F - 6	493.0
F - 7	492.7
F - 8	492.2



GAGE	WATER-SURFACE ELEVATIONS
F-5	493.5
F-6	493.0
F-7	492.7
F-8	492.2

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 11+96.7 TO 29+40.0
 DISCHARGE 7200 CFS
 UNGATED FLOW

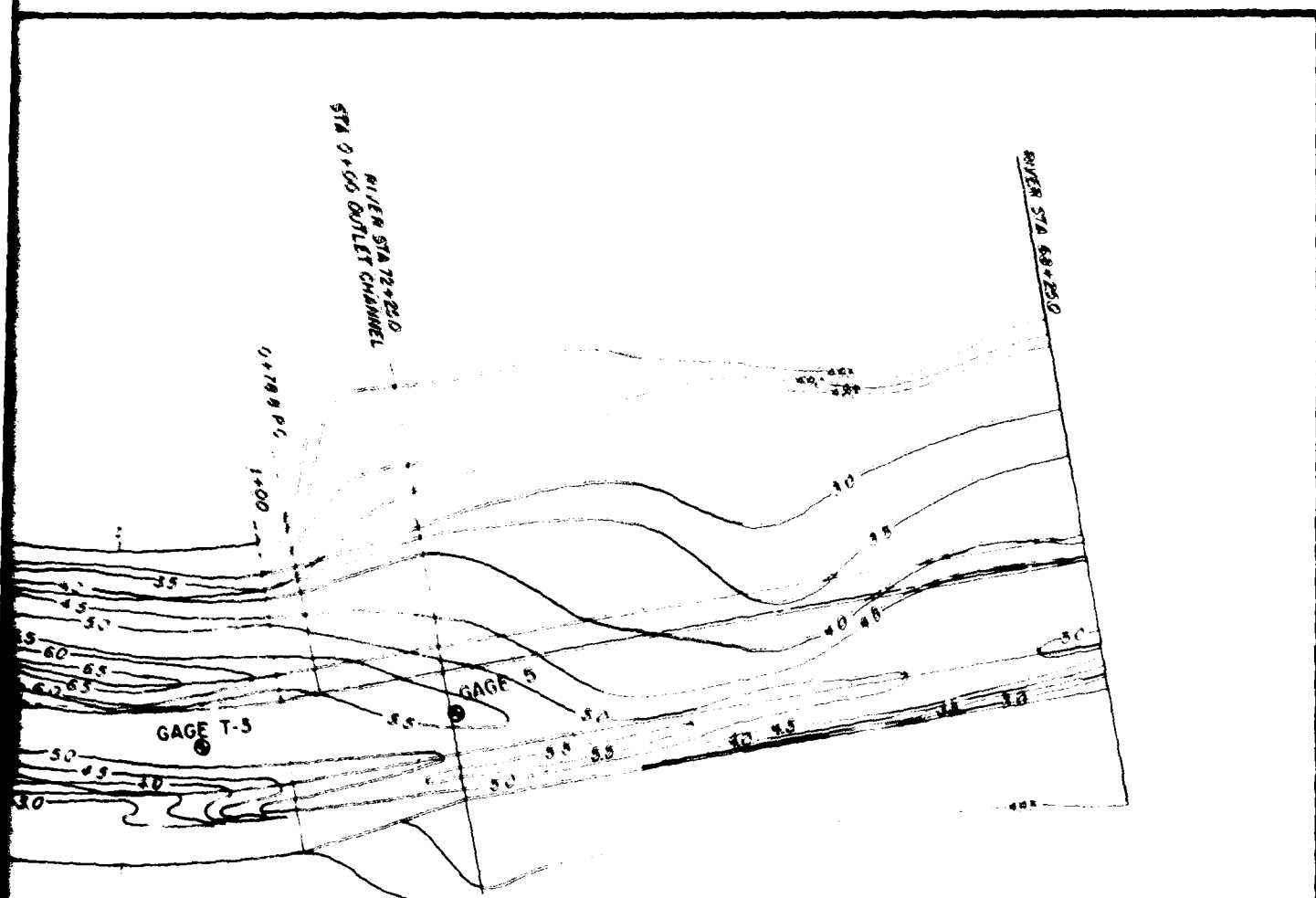




NOTES

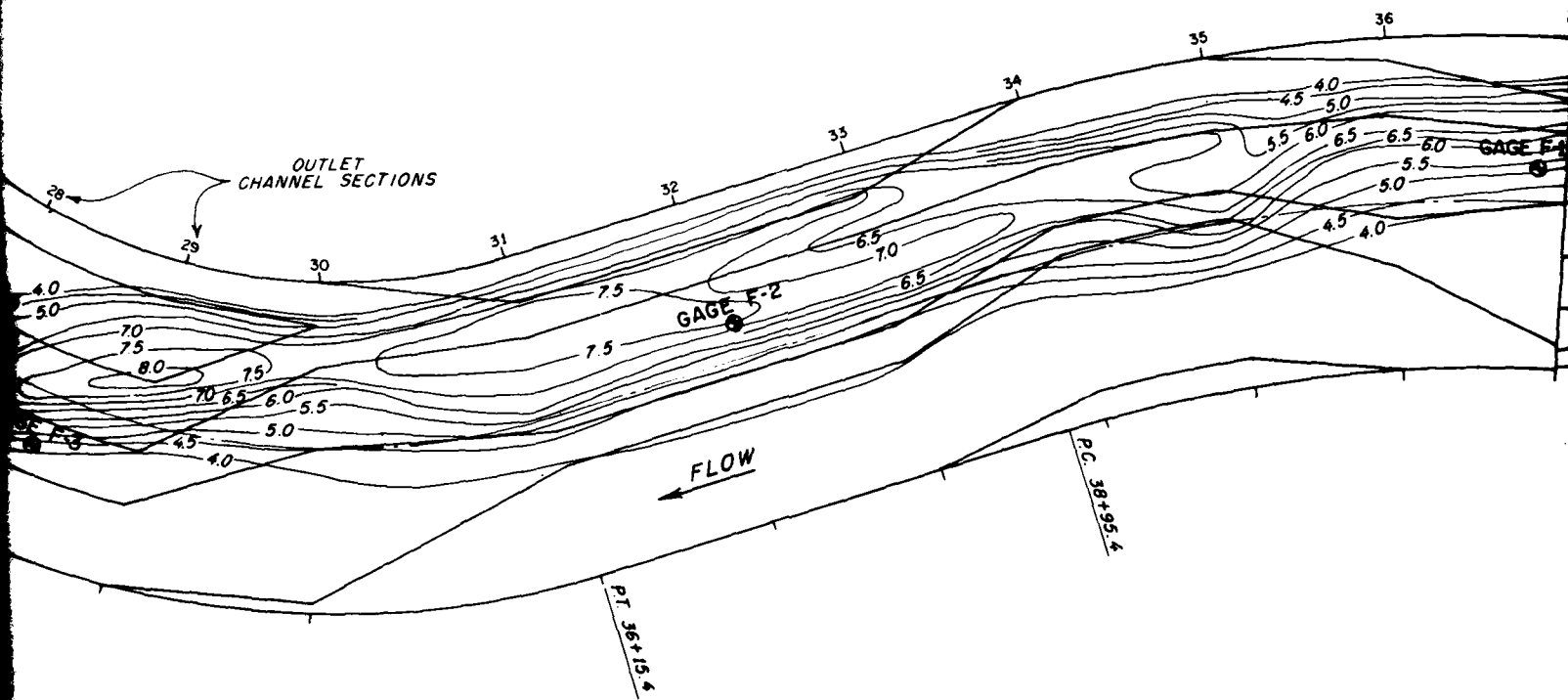
1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.
3. FOR VELOCITIES AT OUTLET STRUCTURE SEE PLATE 36.

GAGE	WATER-SURFACE ELEVATIONS
T-1	491.9
T-2	491.7
T-3	491.5
5	491.4



GAGE	WATER-SURFACE ELEVATIONS
T-1	491.9
T-2	491.7
T-3	491.5
5	491.4

BOTTOM VELOCITIES
PLAN B OUTLET CHANNEL
AND
ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
RIVER STATION 65+94.2 TO CHANNEL STATION 7+44.7
DISCHARGE 7200 CFS
UNGATED FLOW

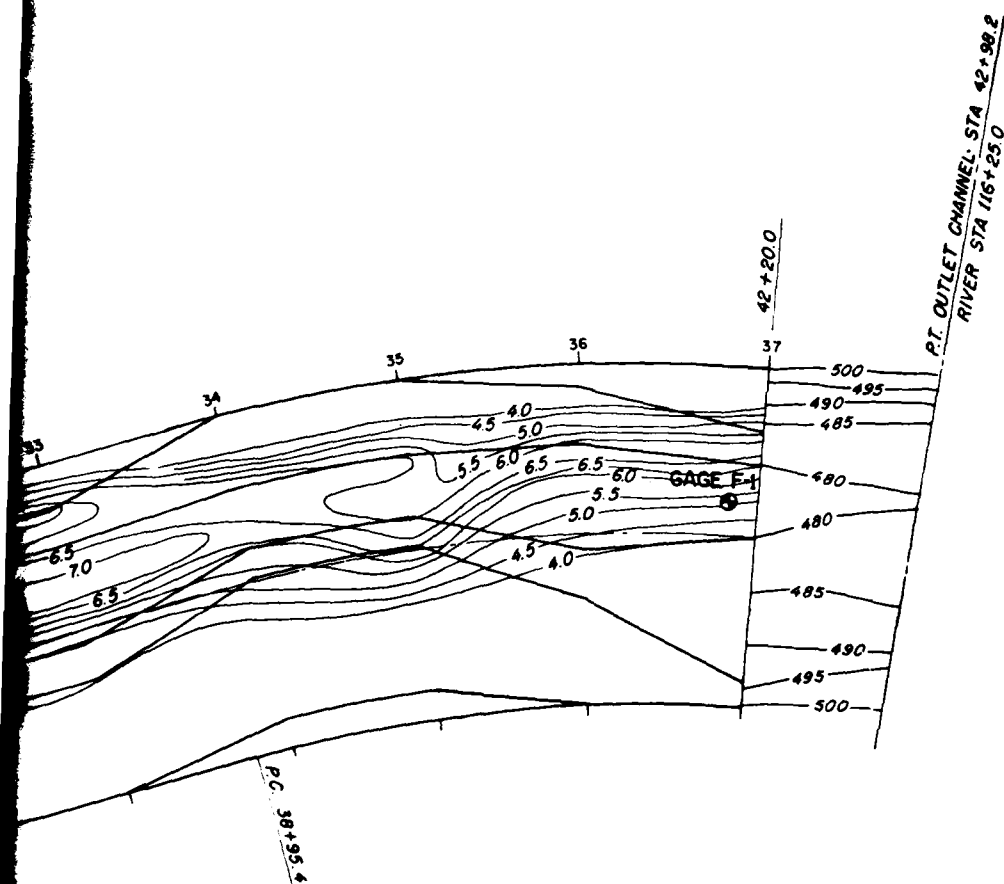


NOTES

1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.

GAGE	WATER-SURFACE ELEVATIONS
F-1	496.3
F-2	495.8
F-3	495.6
F-4	495.3

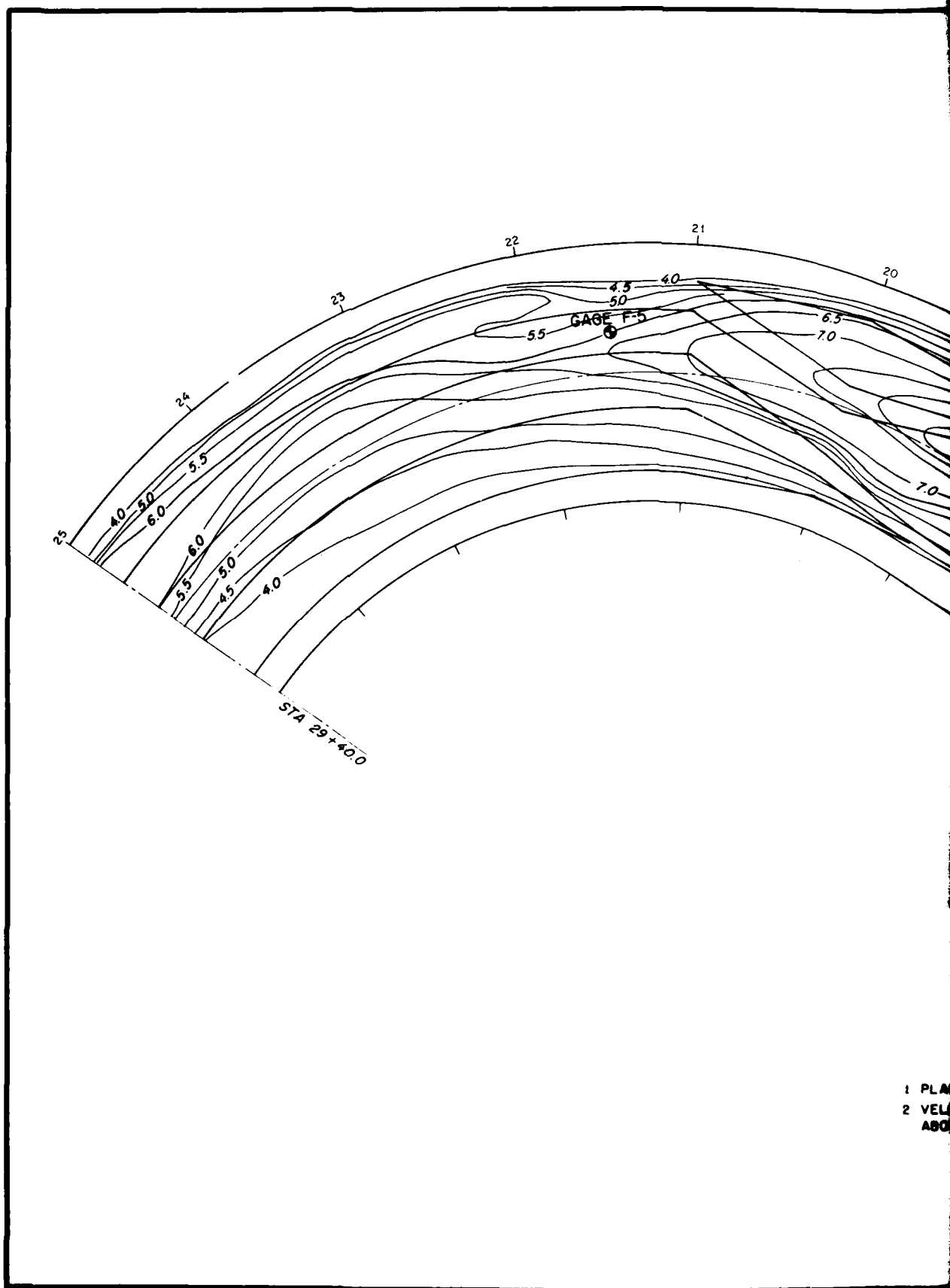
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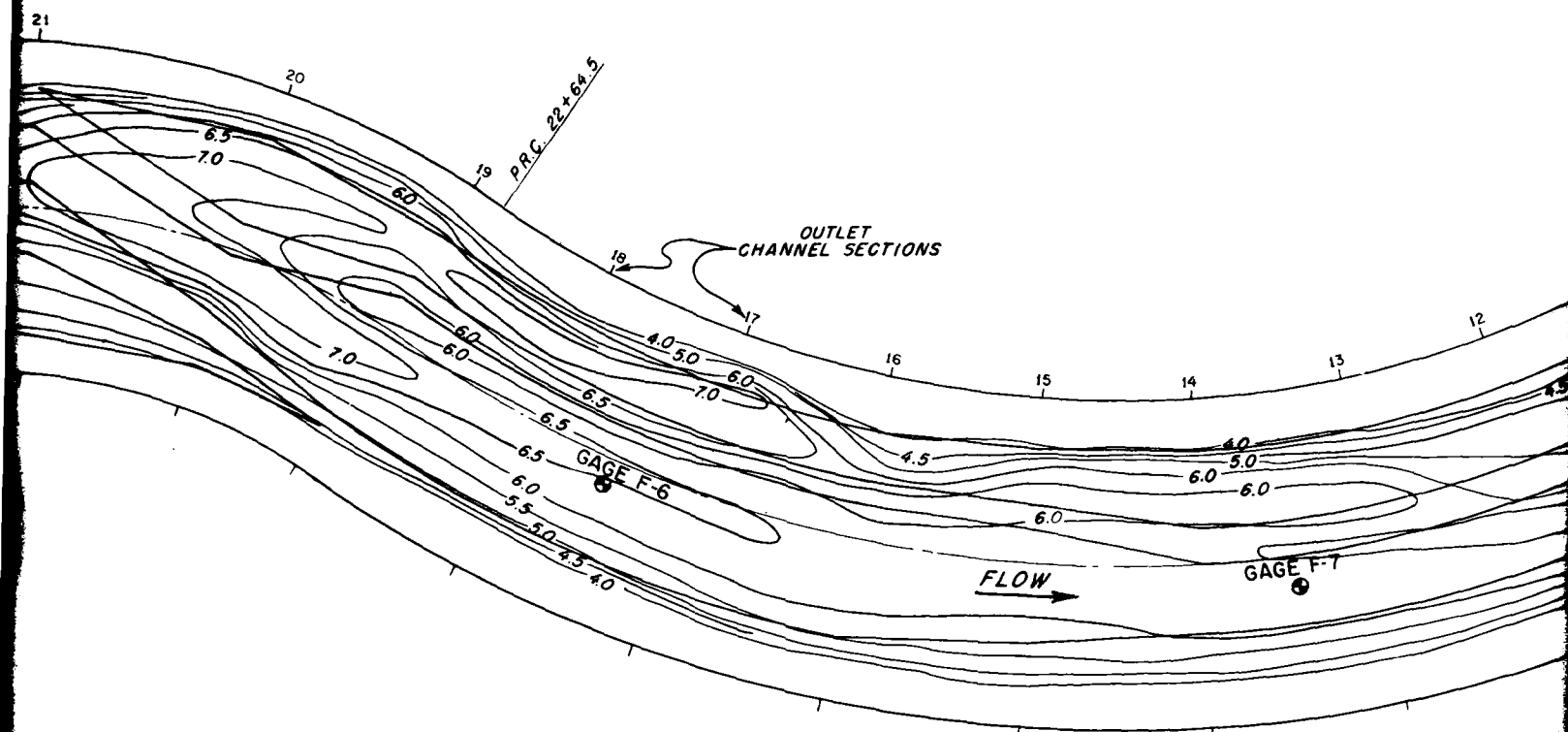
GAGE	WATER-SURFACE ELEVATIONS
F-1	496.3
F-2	495.8
F-3	495.6
F-4	495.3

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 29+40.0 TO 42+20.0
 DISCHARGE 9000 CFS
 UNGATED FLOW

3



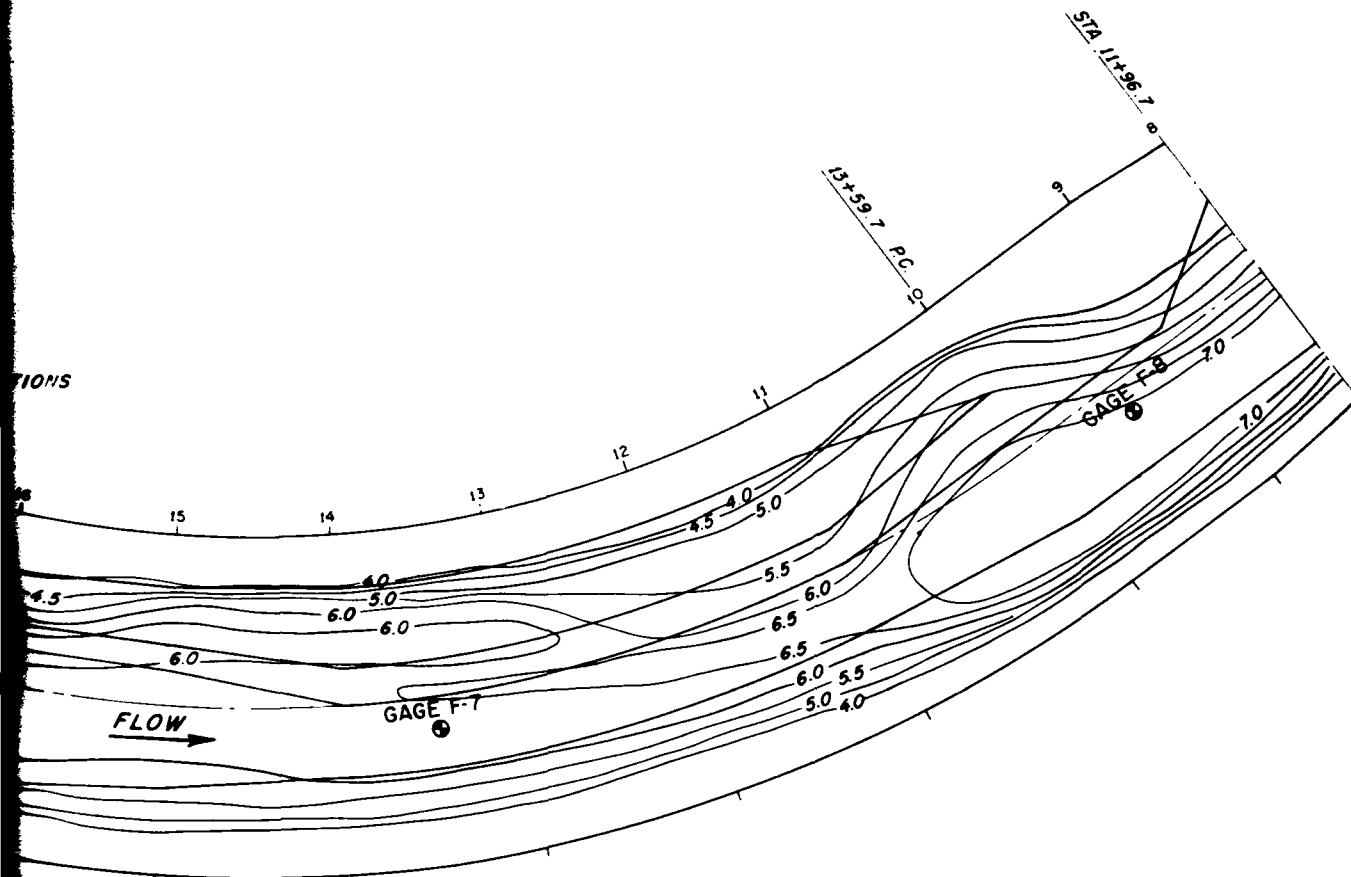
1 PLA
2 VEL
ABO



NOTES

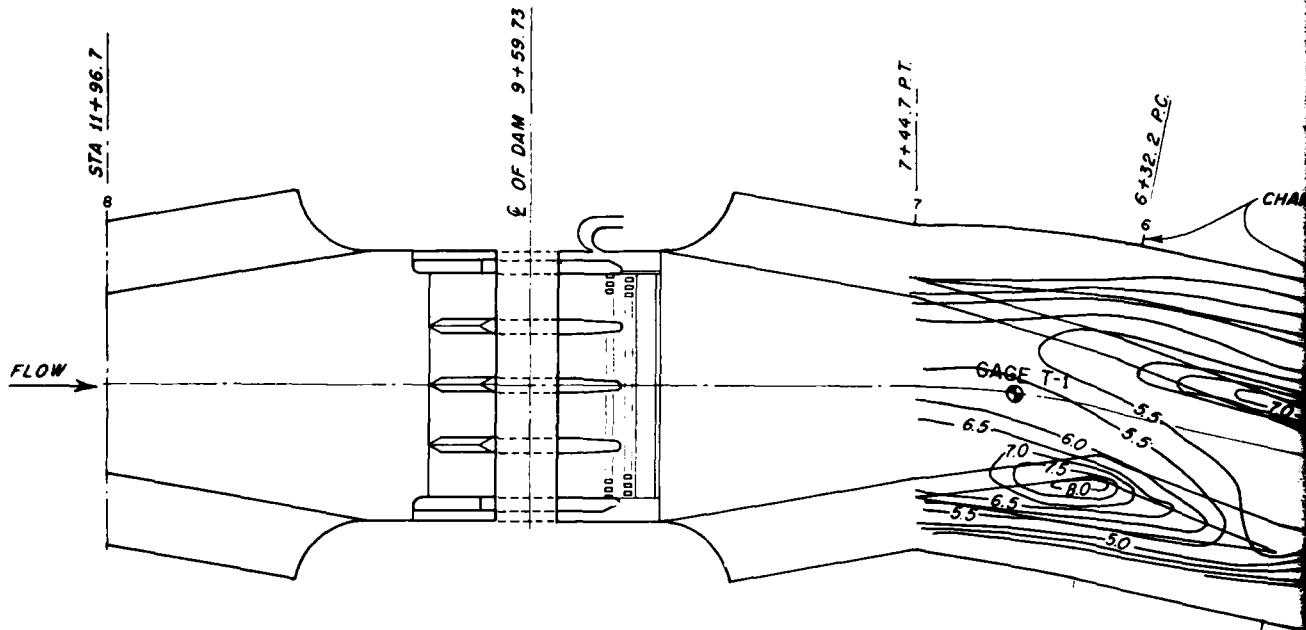
1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM

GAGE	WATER-SURFACE ELEVATIONS
F-5	495.1
F-6	494.4
F-7	494.2
F-8	493.7

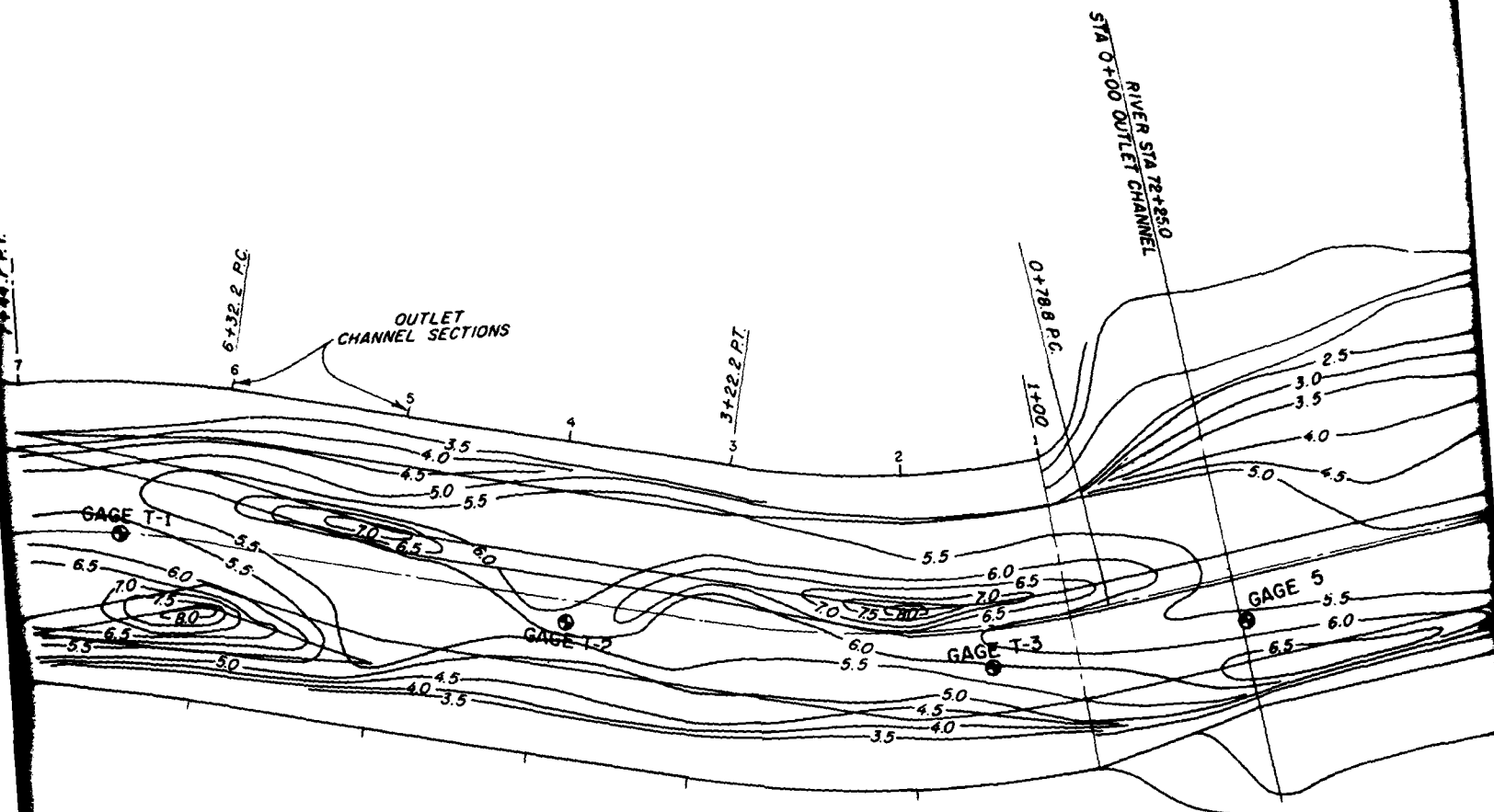


GAGE	WATER-SURFACE ELEVATIONS
F-5	495.1
F-6	494.4
F-7	494.2
F-8	493.7

BOTTOM VELOCITIES
 PLAN B OUTLET CHANNEL
 STATION 11+96.7 TO 29+40.0
 DISCHARGE 9000 CFS
 UNGATED FLOW



- NO
1. PLAN C OUT
 2. VELOCITIES ABOVE BOT
 3. FOR VELOCITY STRUCTURE

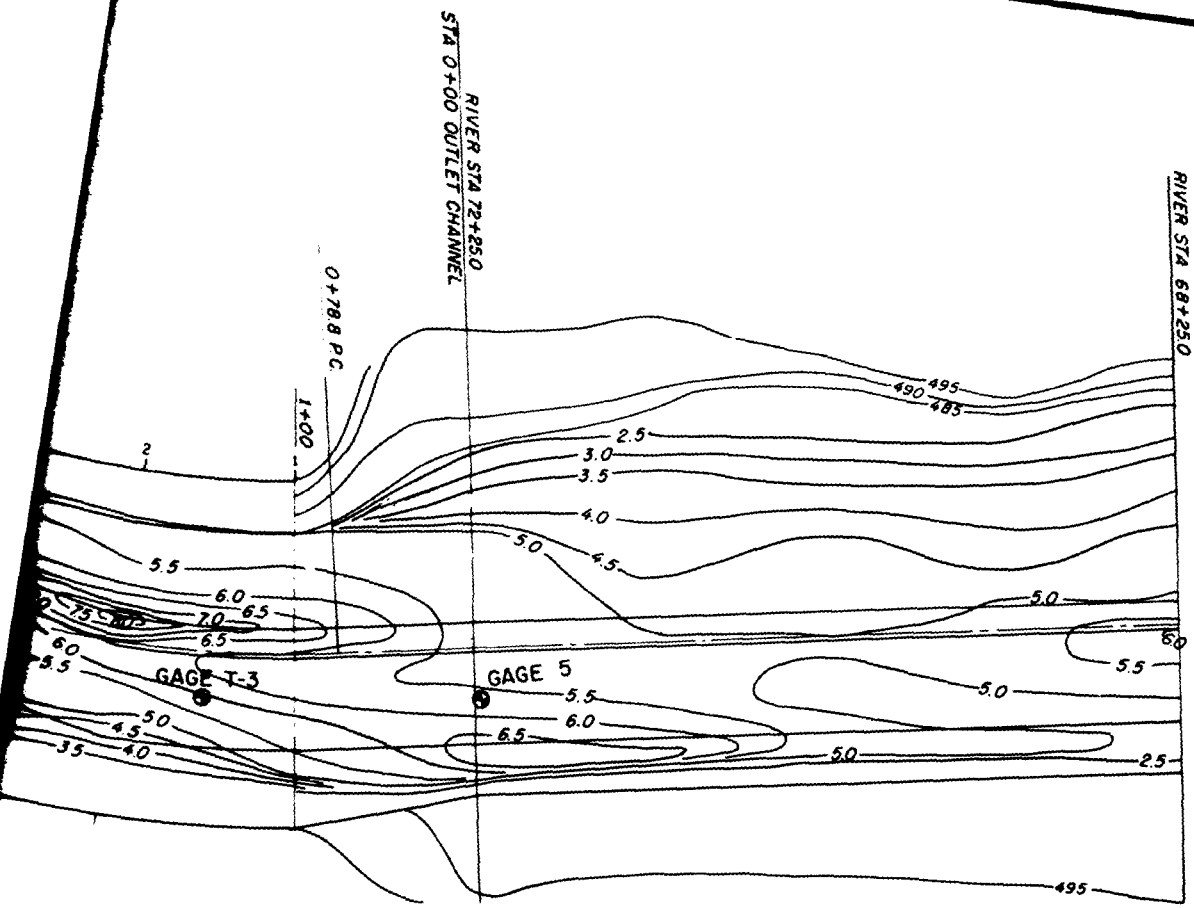


NOTES

1. PLAN C OUTLET STRUCTURE.
2. VELOCITIES IN FPS, 1 TO 1.5 FT. ABOVE BOTTOM.
3. FOR VELOCITIES AT OUTLET STRUCTURE SEE PLATE 37.

GAGE	WATER-SURFACE ELEVATIONS
T-1	493.4
T-2	493.0
T-3	492.8
5	492.7

2

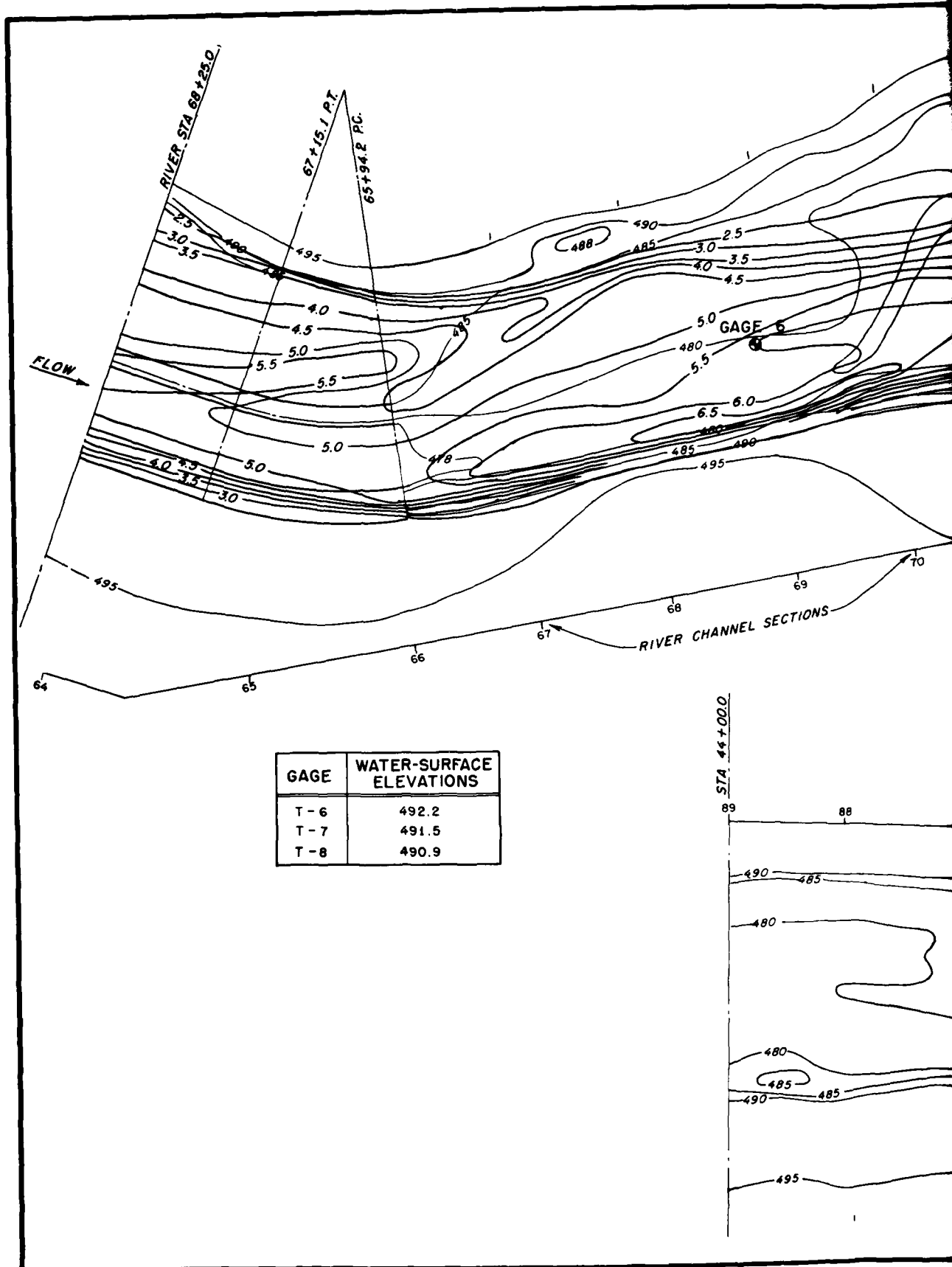


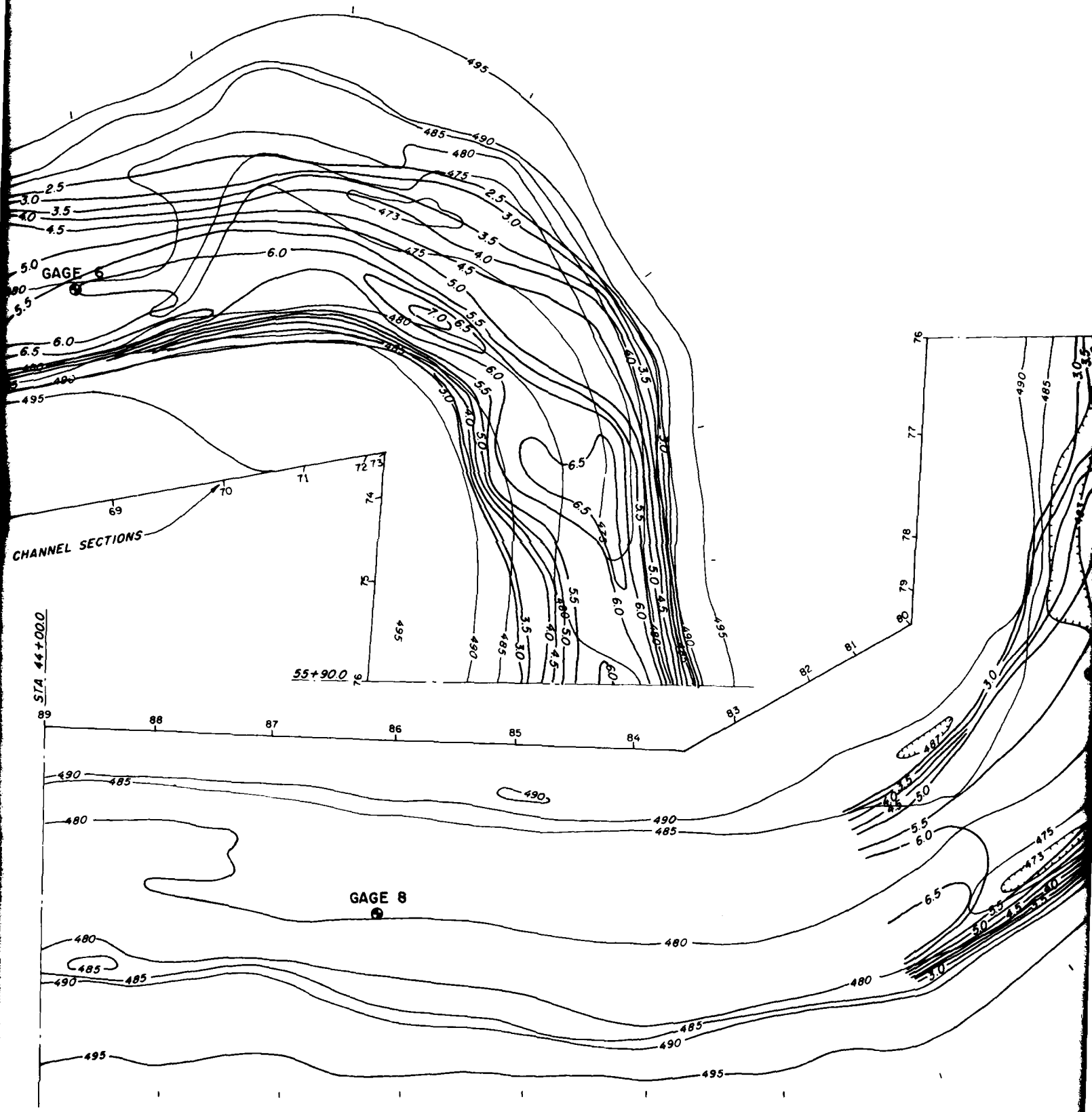
GAGE	WATER-SURFACE ELEVATIONS
T-1	493.4
T-2	493.0
T-3	492.8
5	492.7

BOTTOM VELOCITIES
PLAN B OUTLET CHANNEL
AND
ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
RIVER STATION 68+25.0 TO CHANNEL STATION 7+44.7
DISCHARGE 9000 CFS
UNGATED FLOW

2

1

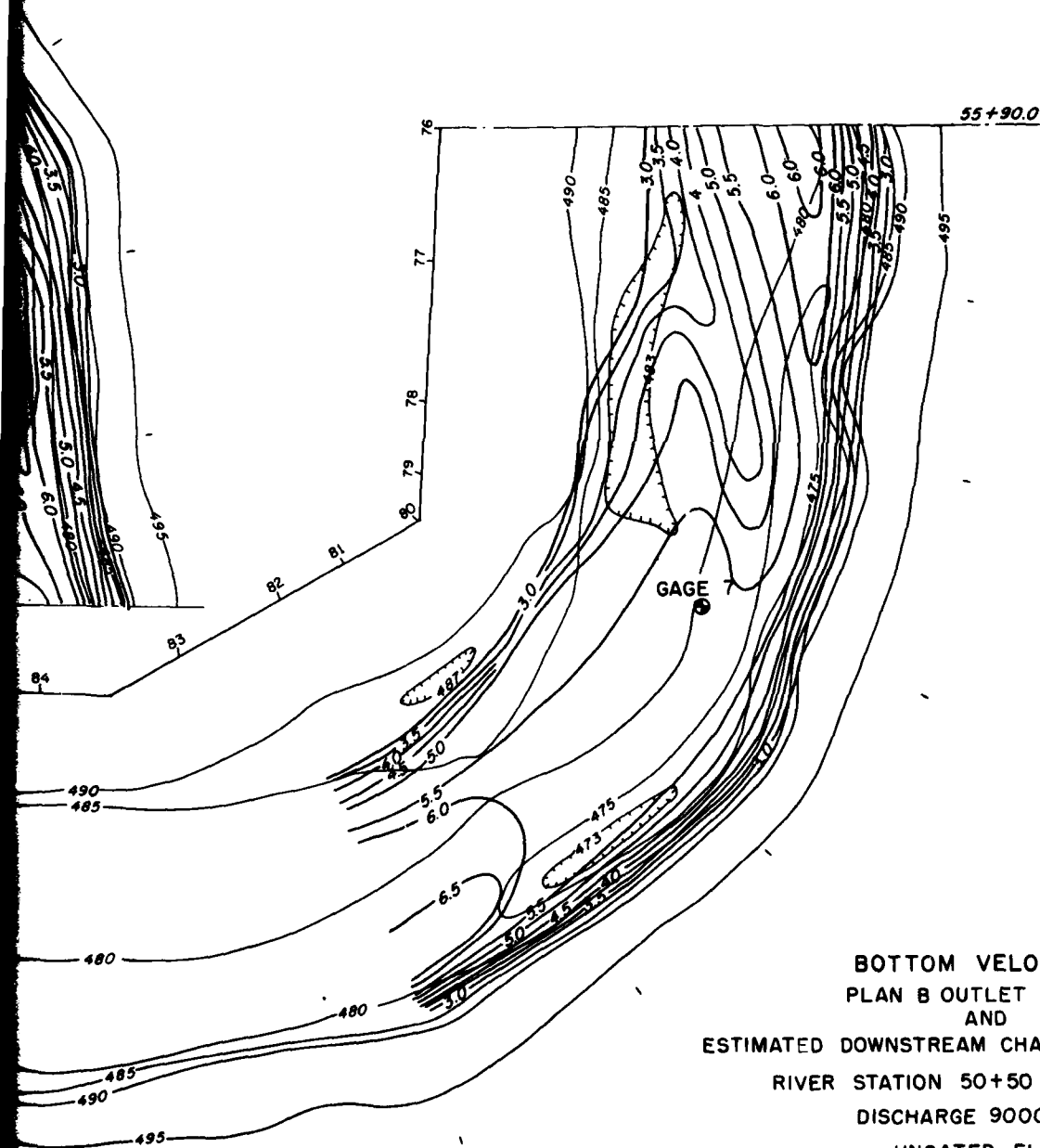




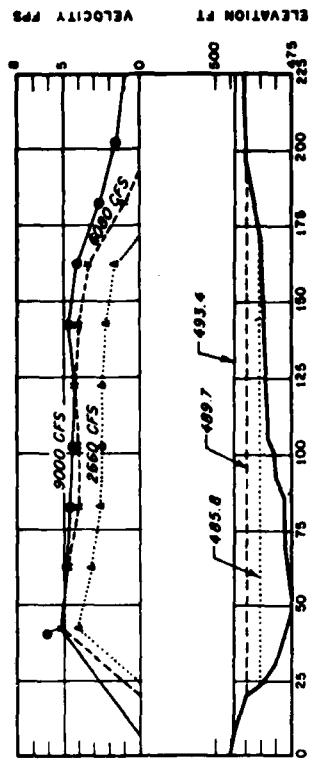
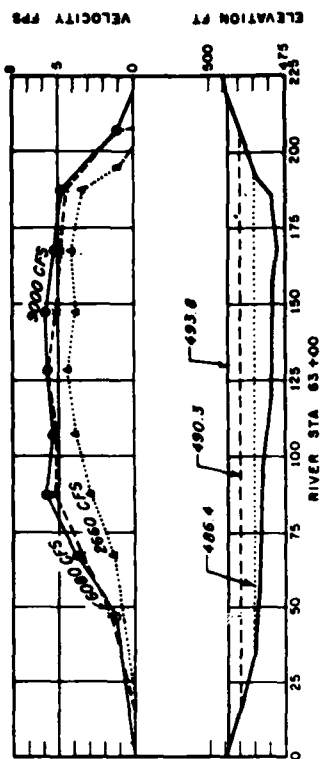
1

2

2

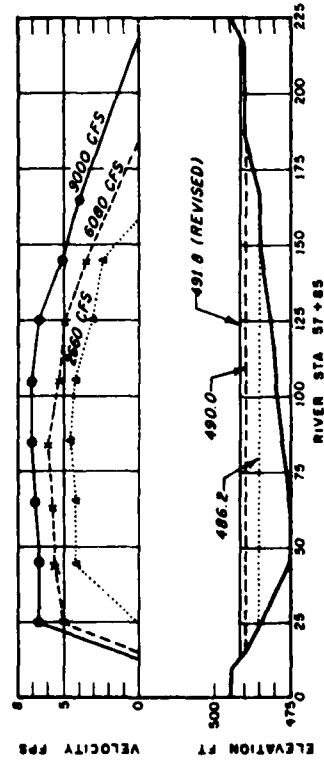
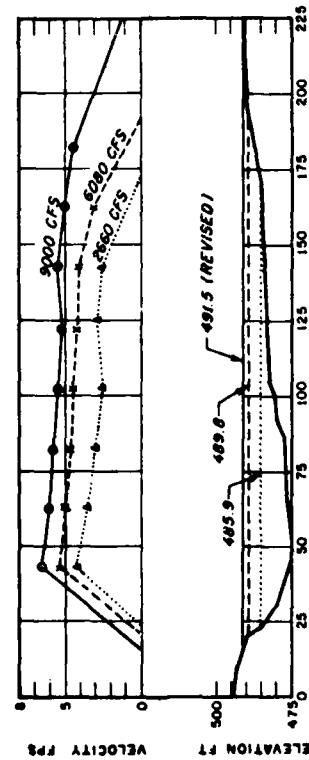
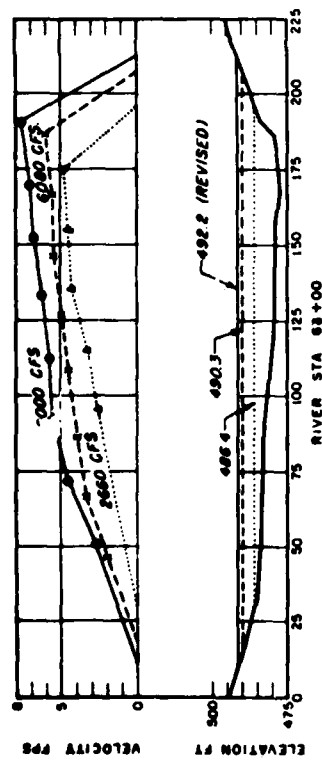


BOTTOM VELOCITIES
PLAN B OUTLET CHANNEL
AND
ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT
RIVER STATION 50+50 TO 68+25
DISCHARGE 9000 CFS
UNGATED FLOW



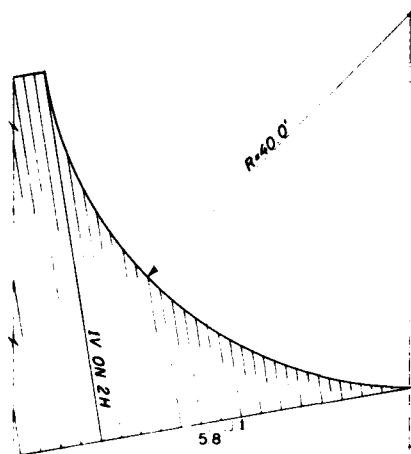
RIVER VERIFICATION

- NOTES
1. VELOCITIES IN FPS AT 0.5-DEPTH
 2. AVERAGE WATER SURFACE ELEVATIONS SHOWN
 3. SECTIONS LOOKING DOWNSTREAM



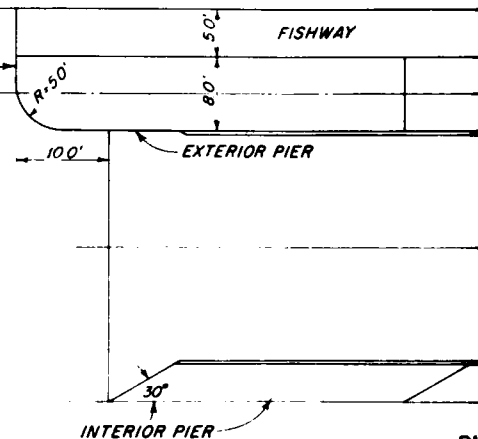
PLAN B OUTLET CHANNEL

WATER SURFACES AND VELOCITIES PLAN B CHANNEL WITH ESTIMATED DOWNSTREAM CHANNEL REALIGNMENT RIVER STATIONS 53+00, 57+85, AND 63+00



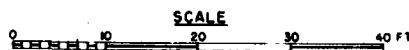
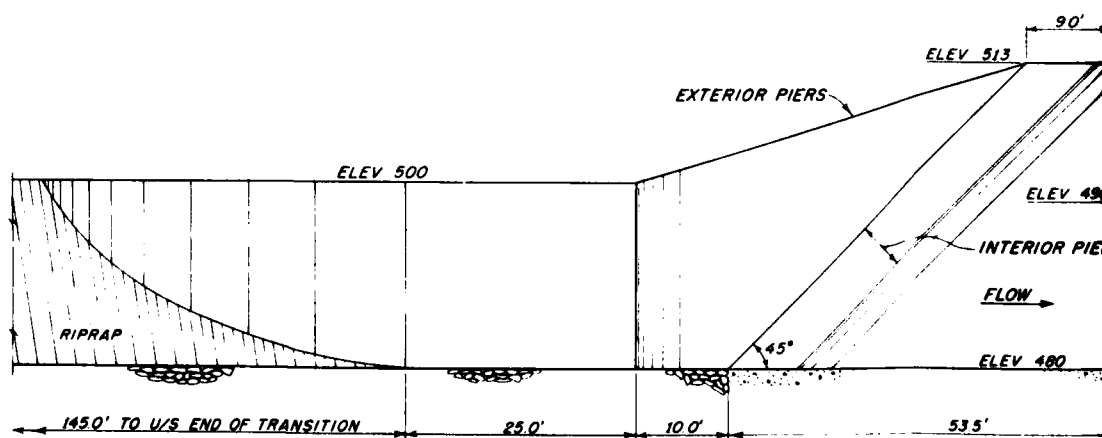
NOTE
FISH LADDER EXIT AND AUXILIARY
WATER SUPPLY CHANNEL NOT
SIMULATED IN MODEL.

RIPRAP ELEV 480

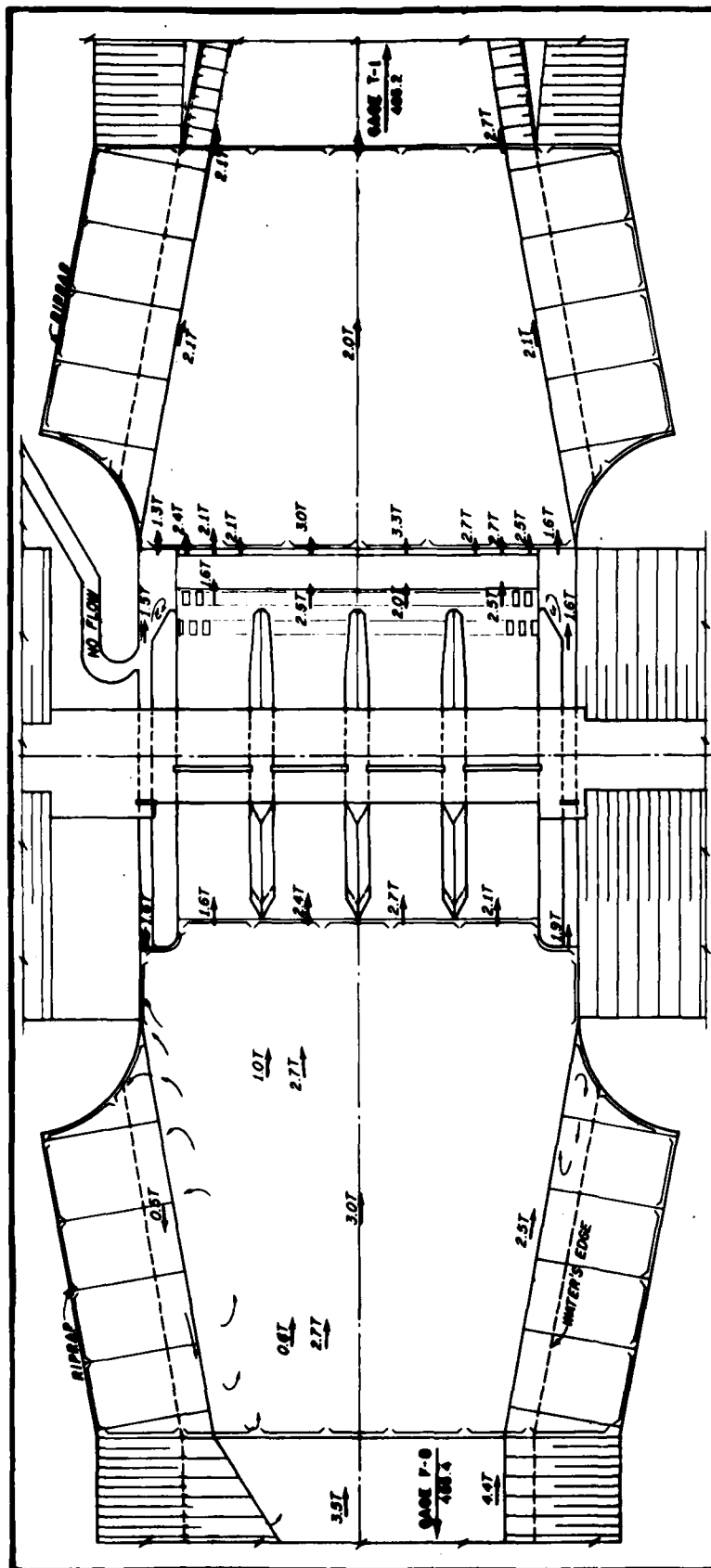


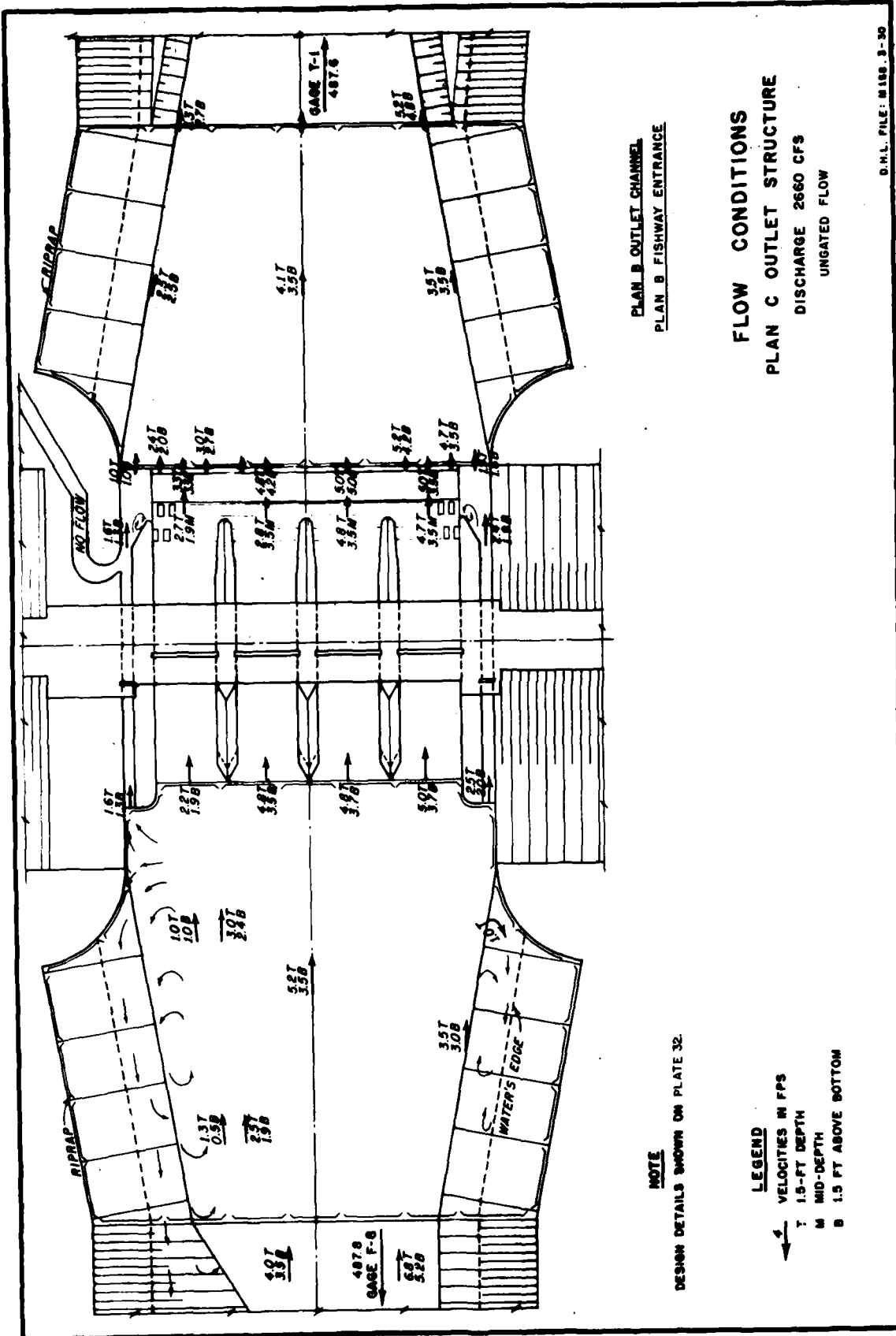
PL

ELEV 527

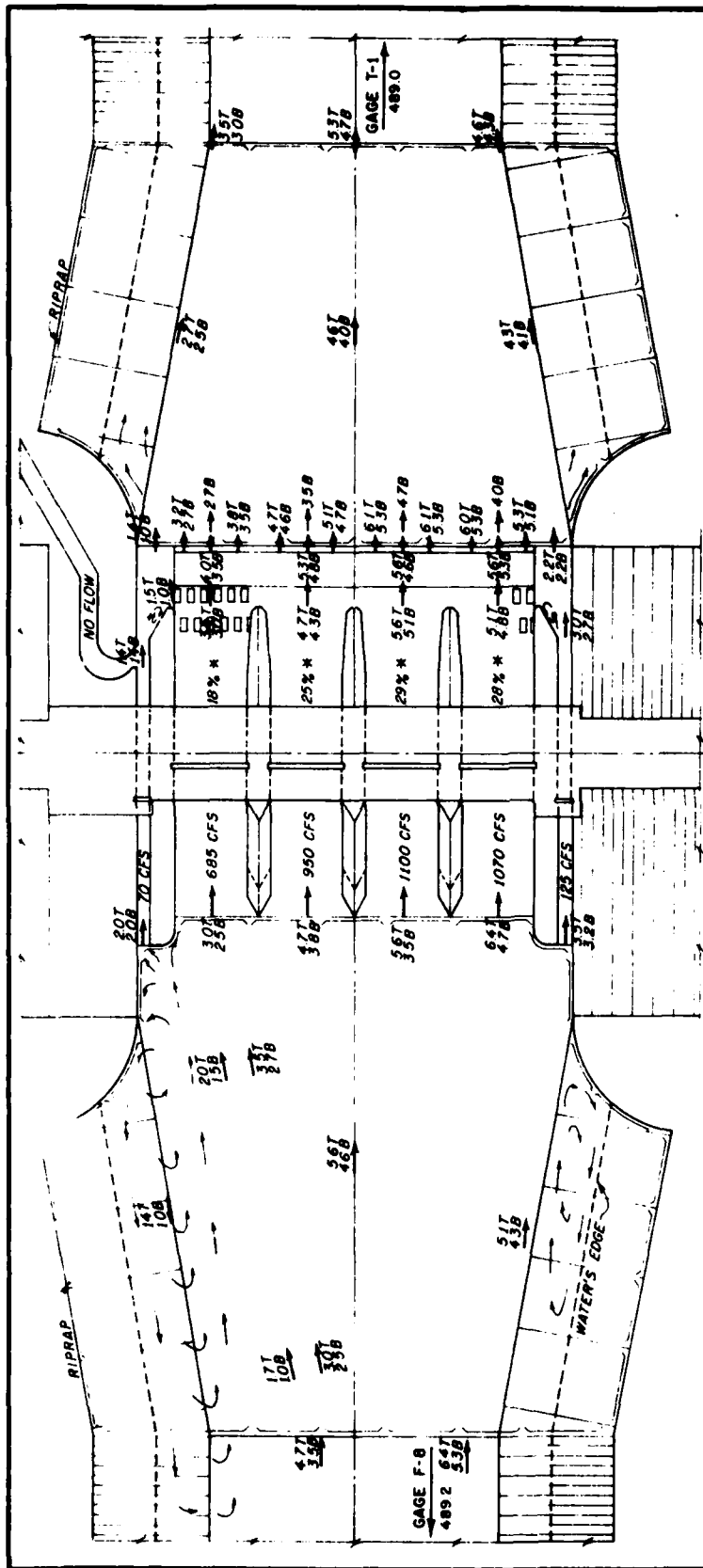


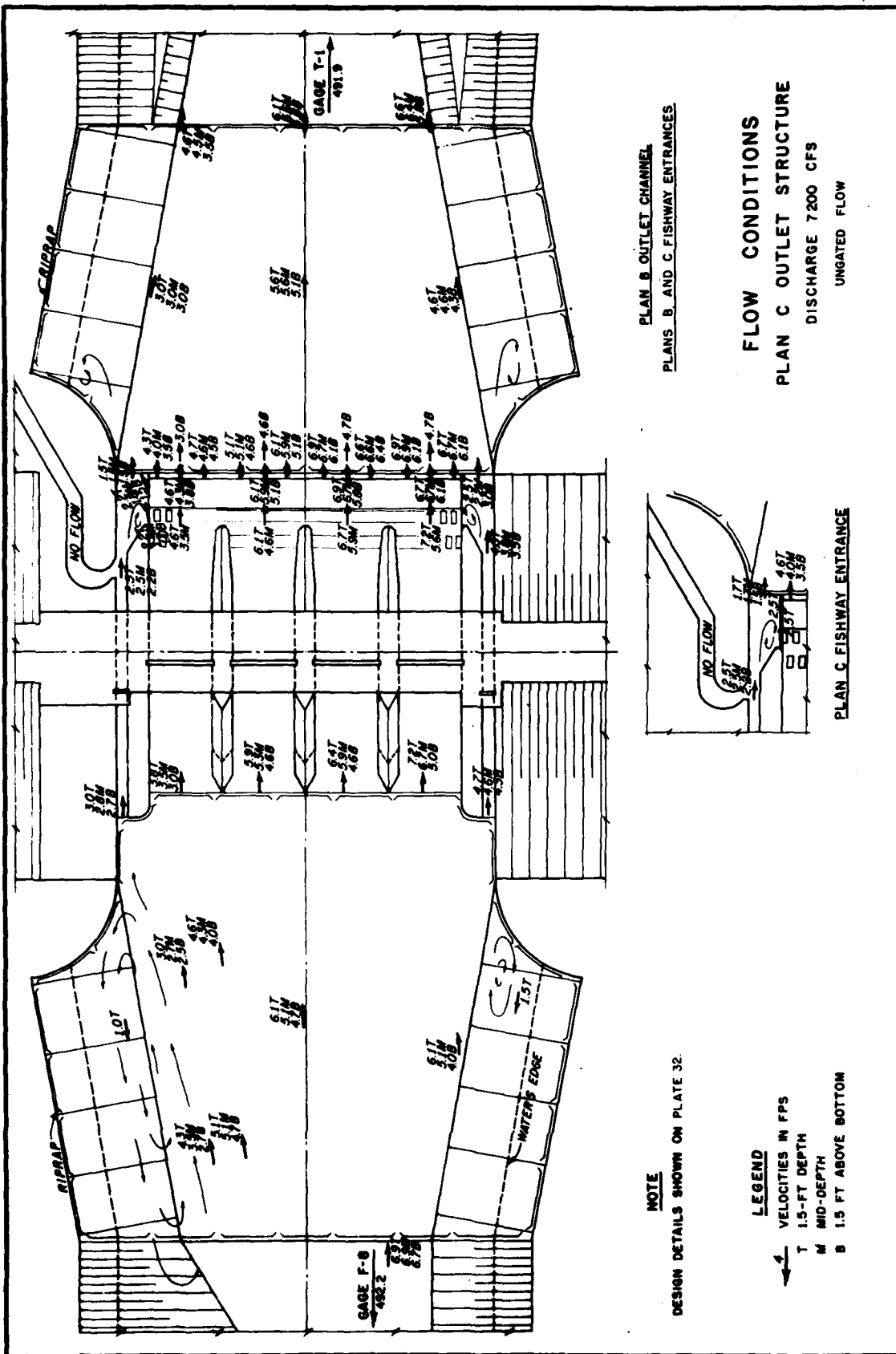
SECTION ALONG C





2

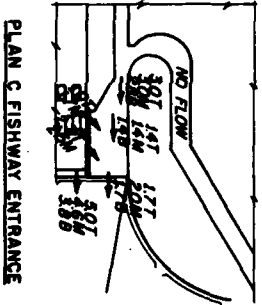
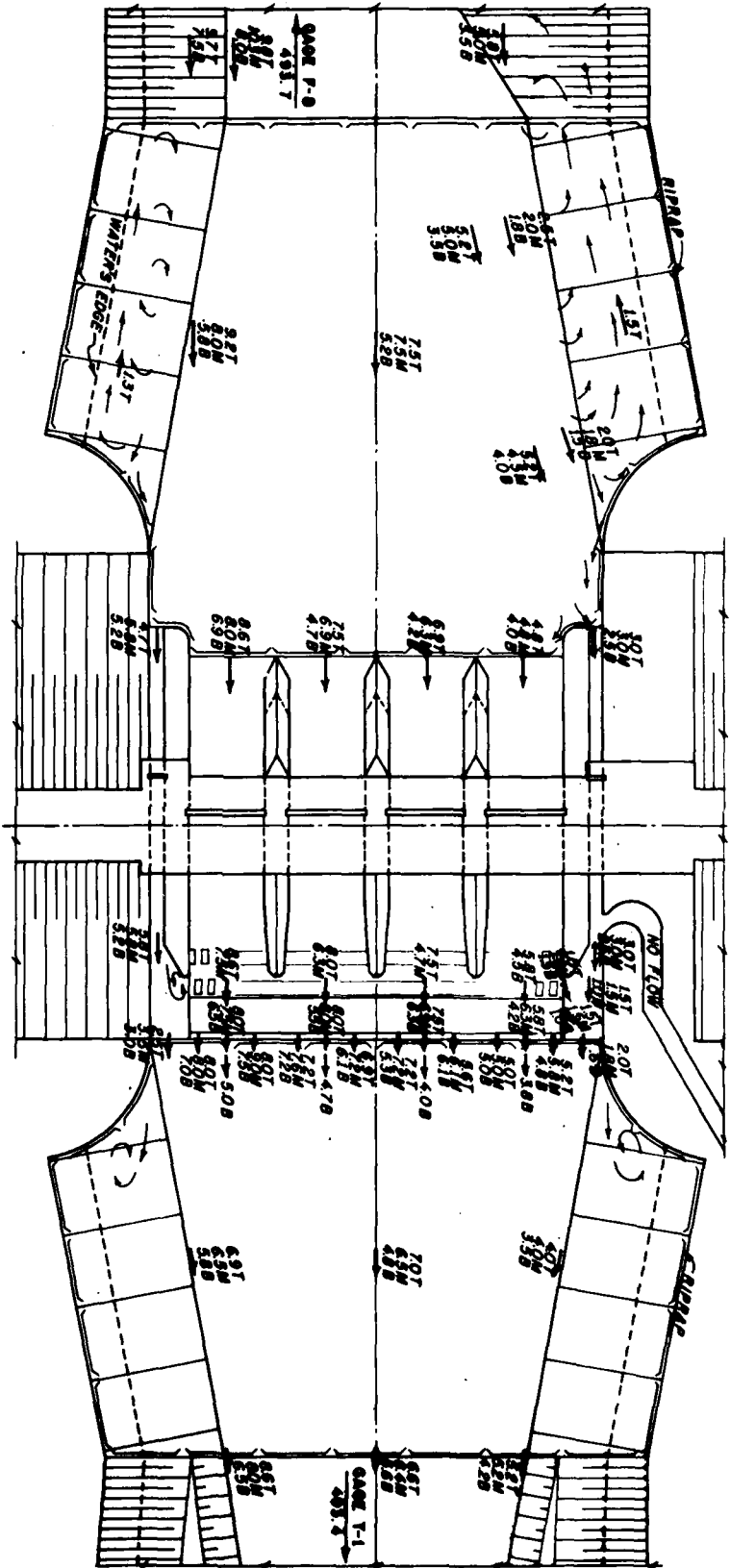




2

NOTE
DESIGN DETAILS SHOWN ON PLATE 32.

LEGEND
VELOCITIES IN FPS
T 1.5-FT DEPTH
M MID-DEPTH
B 1.5 FT ABOVE BOTTOM



PLAN B OUTLET CHANNEL
PLANS B AND C FISHWAY ENTRANCES
FLOW CONDITIONS
PLAN C OUTLET STRUCTURE
DISCHARGE 9000 CFS
UNEATED FLOW

DESIGN DETAILS SHOWN ON PLATE 32.

	↑	VELOCITIES IN FPS
T	1.5-FT DEPTH	
M	MID-DEPTH	
B	1.5 FT ABOVE BOTTOM	

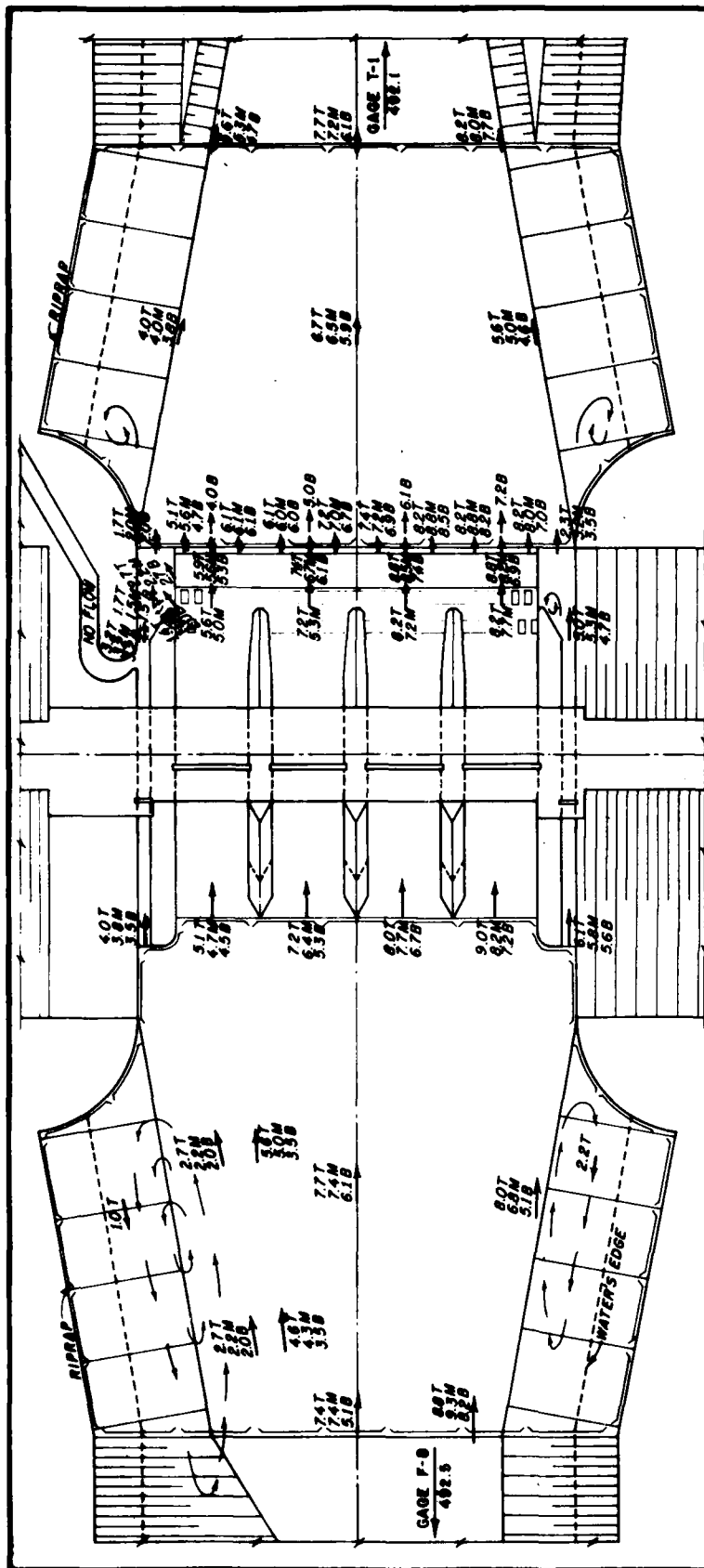
PLANS B AND C FISHWAY ENTRANCES

PLAN C OUTLET STRUCTURE

DISCHARGE 12000 CFS

UNGATED FLOW

PLAN C FISHWAY ENTRANCE



TAILWATER LOWERED 1.3 FT AT GAGE T-1

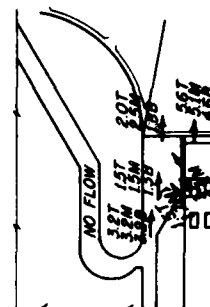
PLAN B OUTLET CHANNEL

PLANS B AND C FISHWAY ENTRANCES

FLOW CONDITIONS PLAN C OUTLET STRUCTURE

DISCHARGE 9000 CFS

UNGATED FLOW



PLAN C FISHWAY ENTRANCE

NOTE

DESIGN DETAILS SHOWN ON PLATE 32.

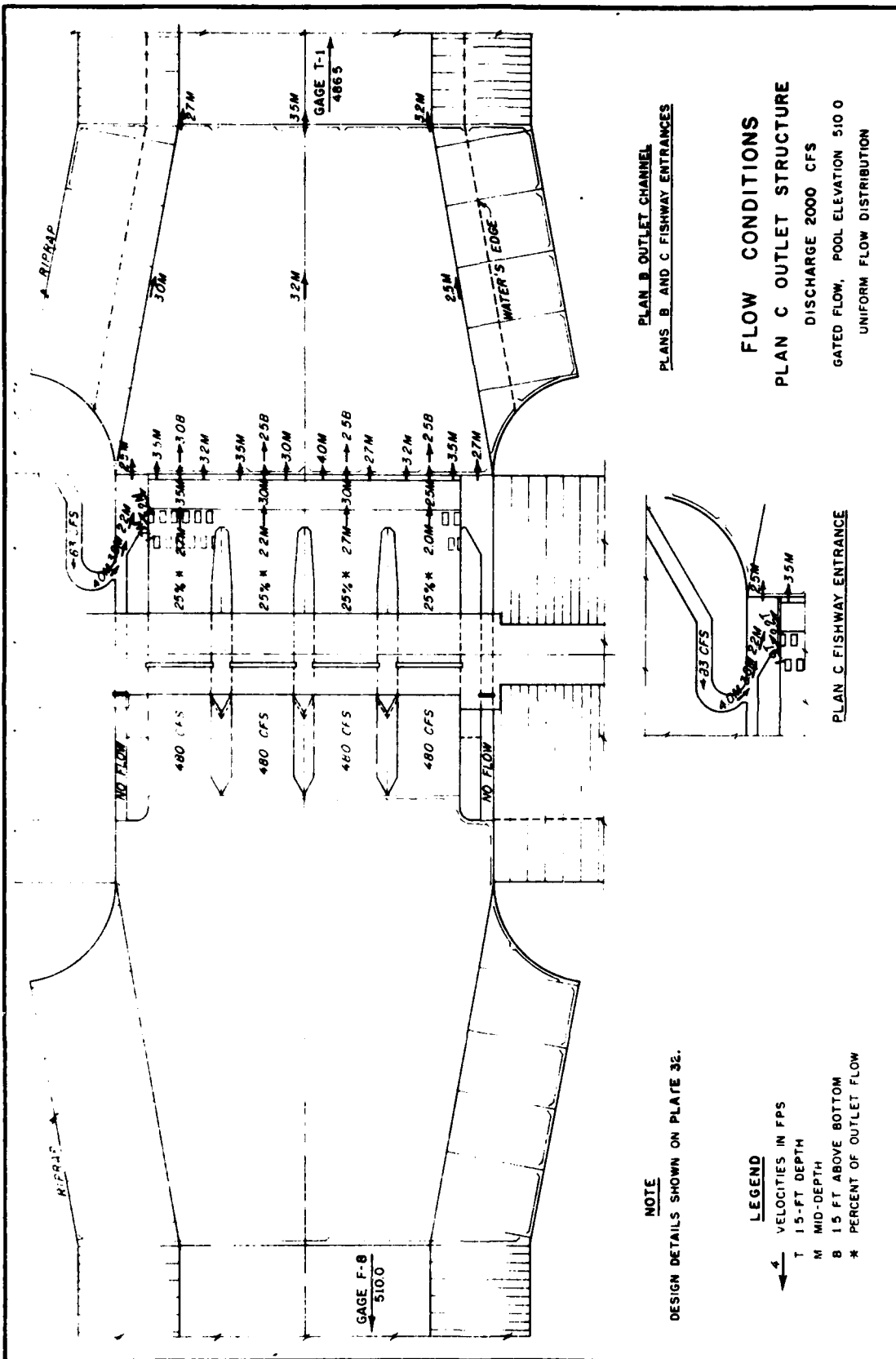
LEGEND

VELOCITIES IN FPS

T 1.5-FT DEPTH

M MID-DEPTH

B 1.5 FT ABOVE BOTTOM



NOTE
DESIGN DETAILS SHOWN ON PLATE 32.

LEGEND

4	VELOCITIES IN FPS
T	15. FT DEPTH
M	MID-DEPTH
B	15 FT ABOVE BOTTOM
X	PERCENT OF OUTLET FLOW

PLAN C FISHWAY ENTRANCE

FLOW CONDITIONS
PLAN C OUTLET STRUCTURE
DISCHARGE 2000 CFS
GATED FLOW, POOL ELEVATION 510.0
UNGATED FLOW DISTRIBUTION

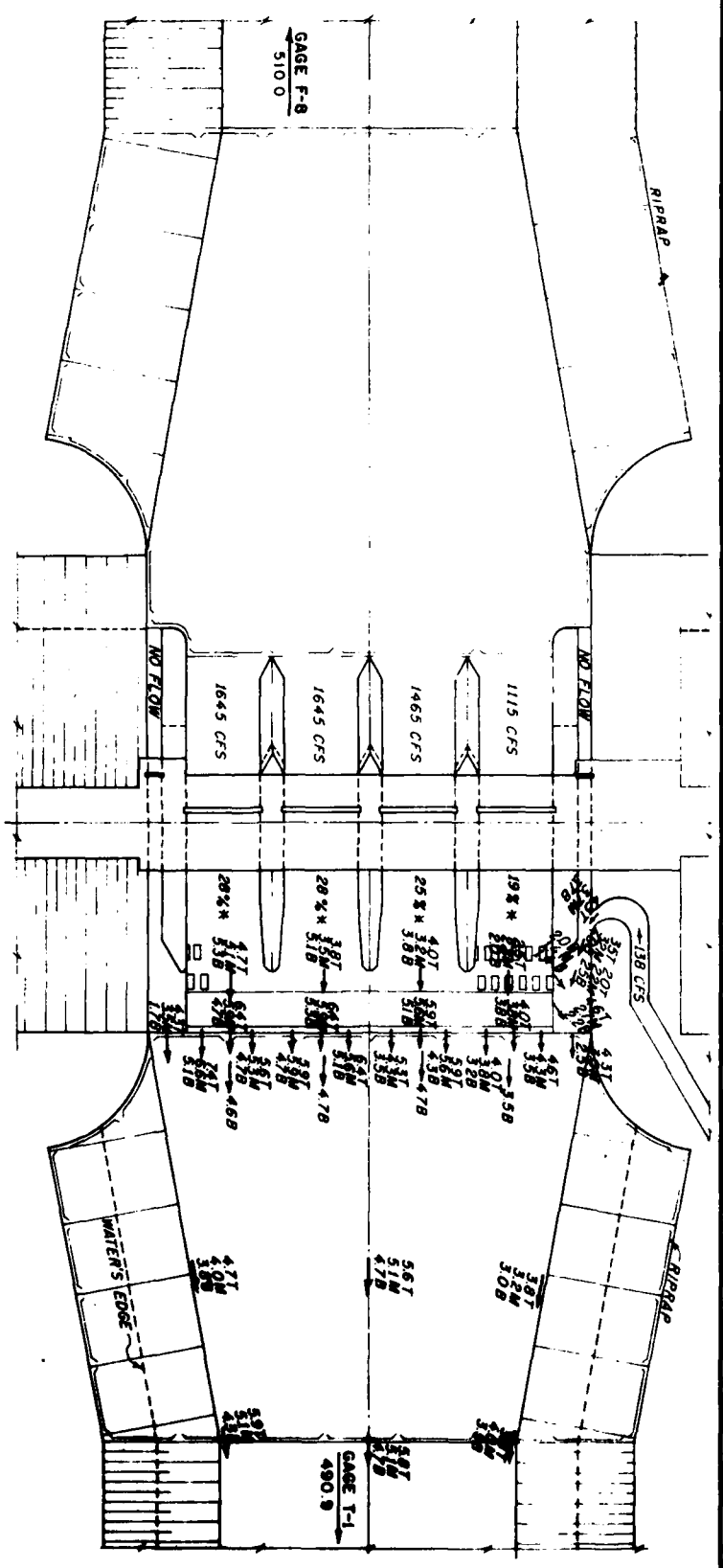
PLAN B OUTLET CHANNEL

FLOW CONDITIONS
PLAN C OUTLET STRUCTURE

GATED FLOW, POOL ELEVATION 510.0

NOTE
DESIGN DETAILS SHOWN ON PLATE 32.

LEGEND
VELOCITIES IN FPS
T 1.5-FT DEPTH
M MID-DEPTH
B 1.5 FT ABOVE BOTTOM
* PERCENT OF OUTLET FLOW



PLAN B, OUTLET CHANNEL
PLANS B AND C FISHWAY ENTRANCES

FLOW CONDITIONS
PLAN C OUTLET STRUCTURE
DISCHARGE 6000 CFS
GATED FLOW, POOL ELEVATION 510.0
UNGATED FLOW DISTRIBUTION



LEGEND

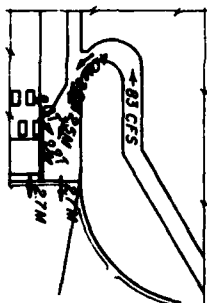
VELOCITIES IN FPS

T 1.5-FT DEPTH

M MID-DEPTH

B 1.5 FT ABOVE BOTTOM

* PERCENT OF OUTLET FLOW



PLAN B OUTLET CHANNEL,
PLANS B AND C FISHWAY ENTRANCES

FLOW CONDITIONS

PLAN C OUTLET STRUCTURE

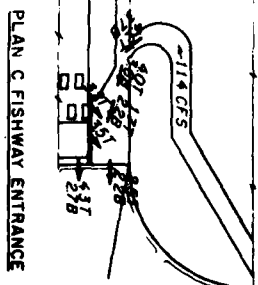
DISCHARGE 2000 CFS

GATED FLOW, POOL ELEVATION 510.0

LINEAR FLOW DISTRIBUTION

NOTE
DESIGN DETAILS SHOWN ON PLATE 32.

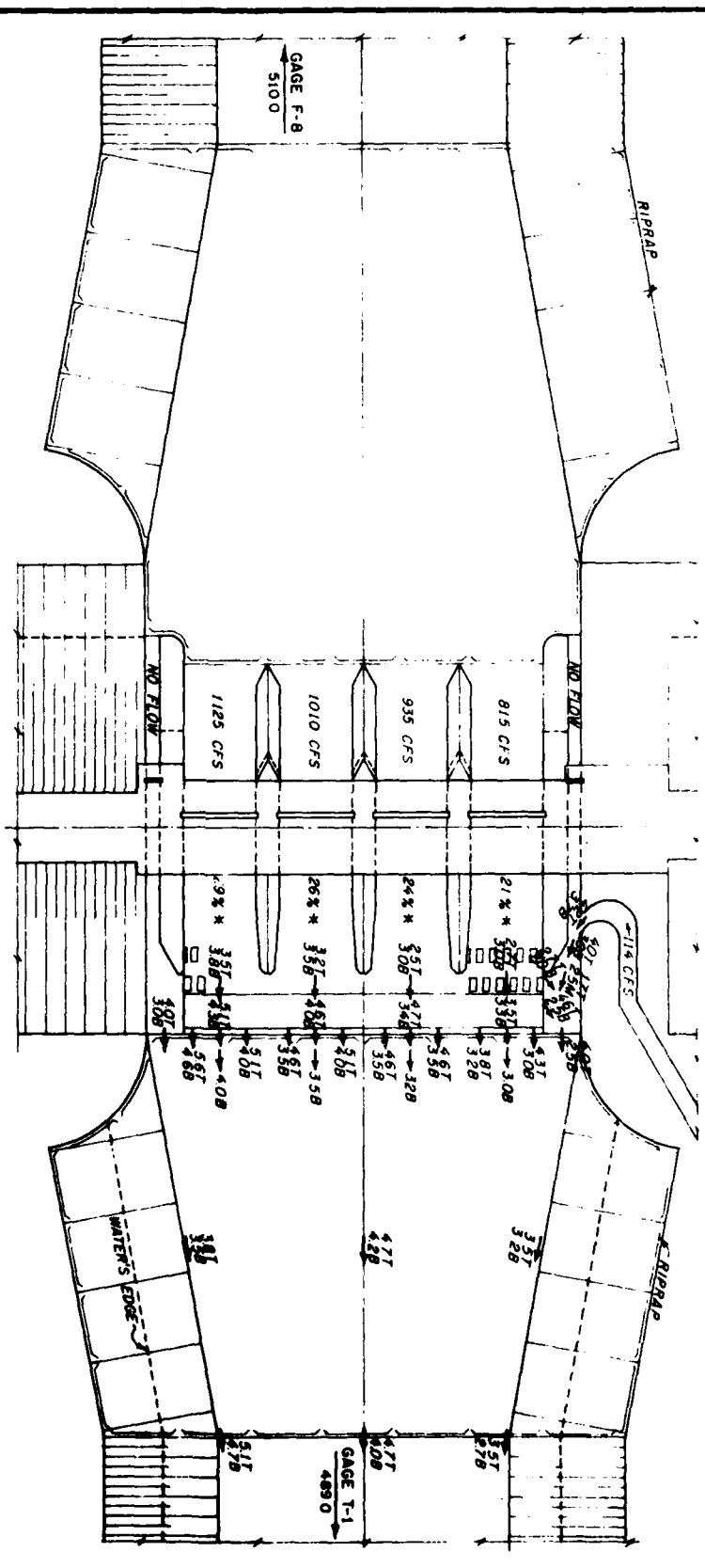
LEGEND
T 15-FT DEPTH
M MID-DEPTH
B 1.5 FT ABOVE BOTTOM
* PERCENT OF OUTLET FLOW



PLAN C FISHWAY ENTRANCE

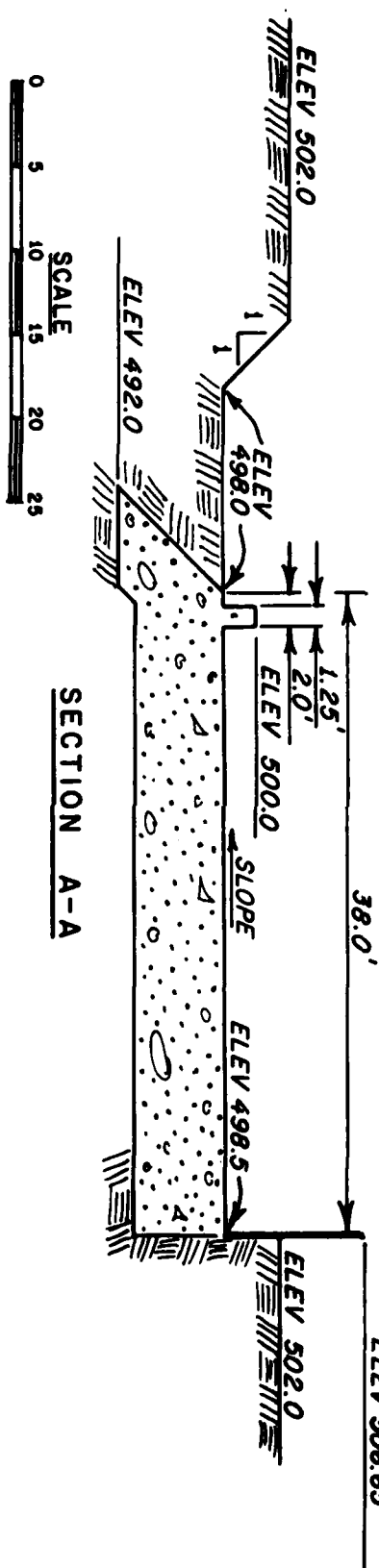
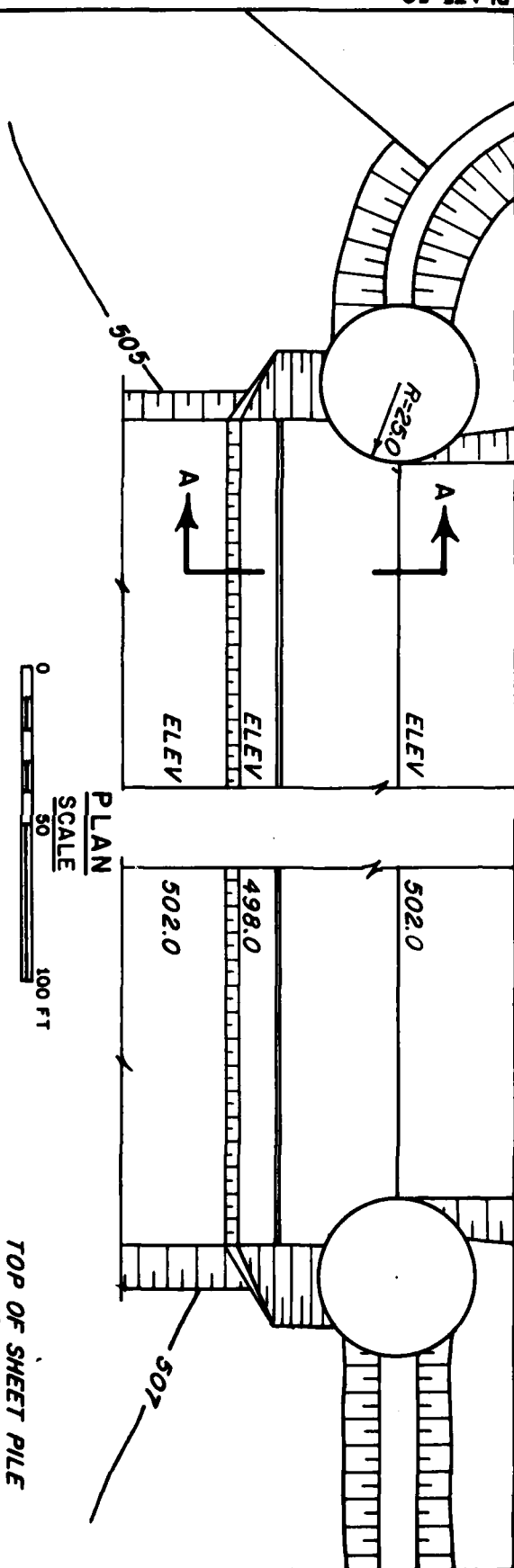
FLOW CONDITIONS
PLAN C OUTLET STRUCTURE
DISCHARGE 4000 CFS
GATED FLOW, POOL ELEVATION 510.0
LINEAR FLOW DISTRIBUTION

PLANS B AND C FISHWAY ENTRANCES

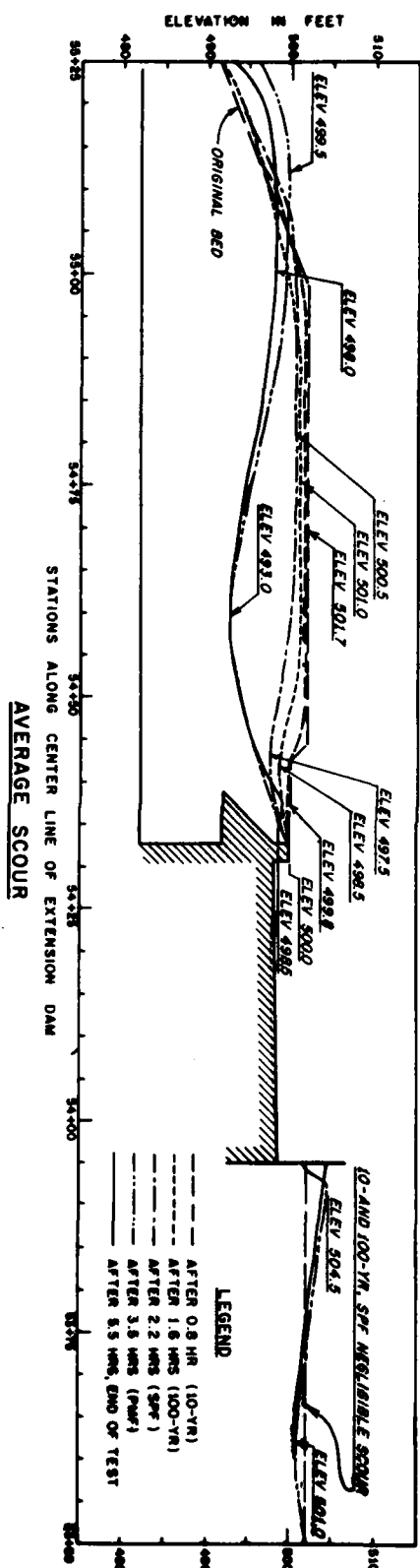


PLAN B OUTLET CHANNEL

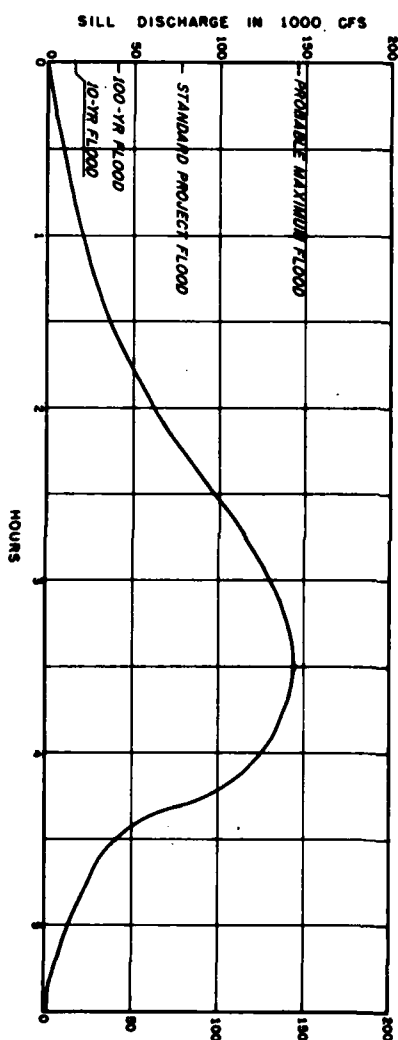
DIVERSION CHANNEL DETAILS OF OVERFLOW SILLS



DIVERSION CHANNEL
OVERFLOW SILL AND BASIN
PLAN 8

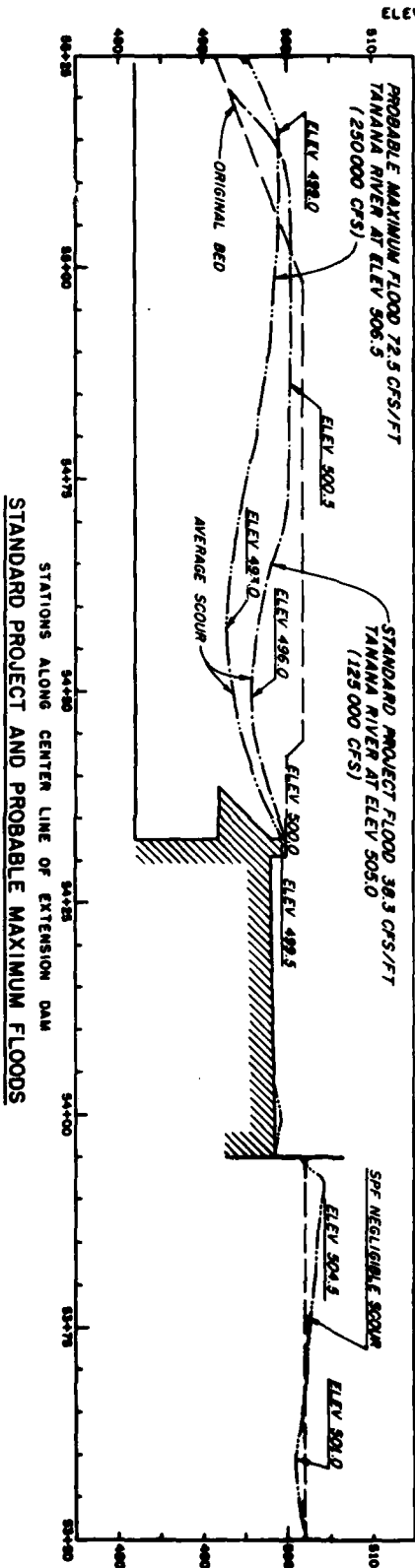
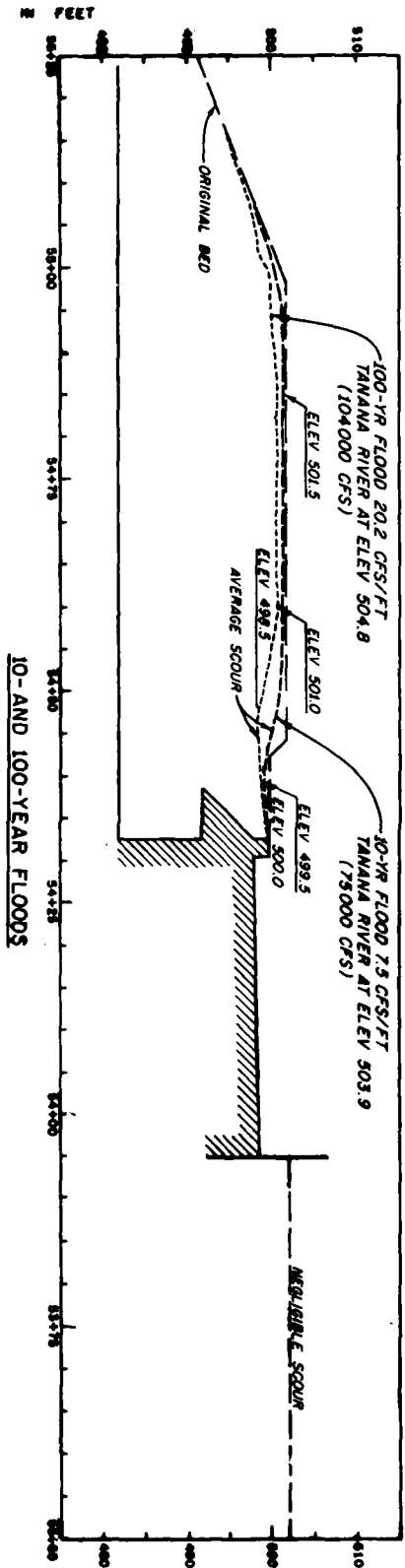


NOTE
DETAILS OF OVERFLOW SILL SHOWN ON PLATE 49.



PROBABLE MAXIMUM FLOOD HYDROGRAPH

SCOUR PROFILES
SHEET PILE OVERFLOW SKILL
WITH PLAN B BASIN
LOW TAILWATER



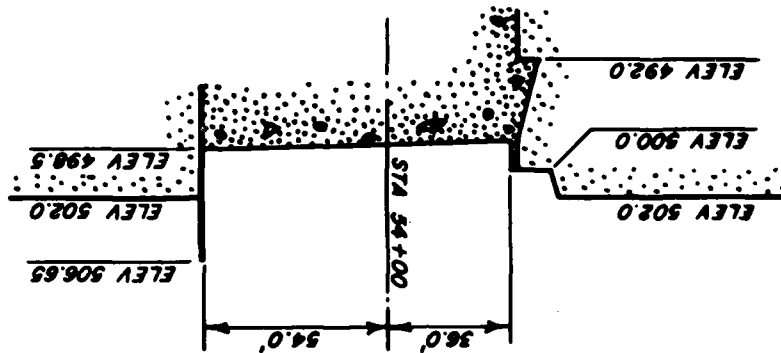
NOTE

DETAILS OF OVERFLOW SILL SHOWN ON PLATE 49.

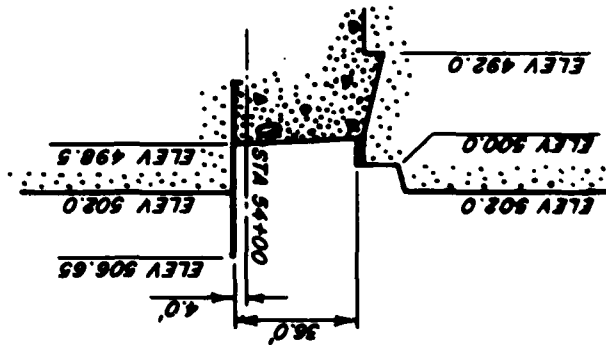
SCOUR PROFILES
SHEET PILE OVERFLOW SILL
WITH PLAN B BASIN
HIGH TAILWATER

SHEET PILE OVERFLOW SILL
WITH PLAN B BASIN

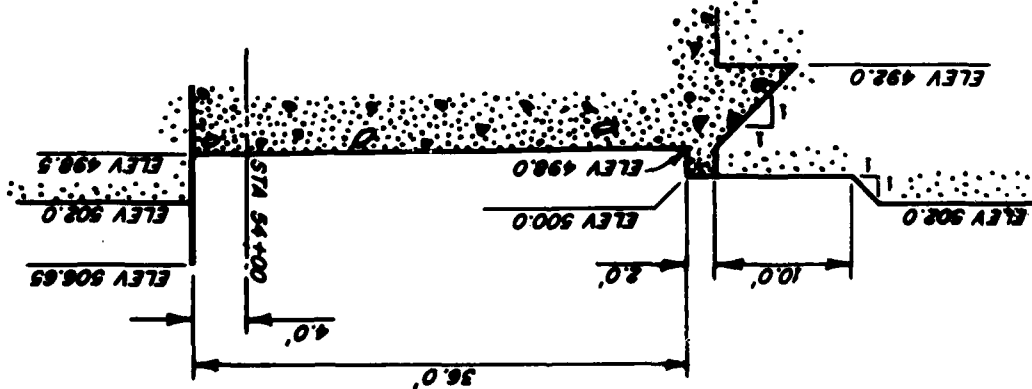
DISTORTED, SCALE 1:100H, 1:25V



SCALE 1:100H, 1:25V



UNDISTORTED, SCALE 1:12



AD-A147 036

MOOSE CREEK DAM OUTLET WORKS AND DIVERSION CHANNEL
CHENA RIVER LAKES PROJ..(U) ARMY ENGINEER DIV NORTH
PACIFIC BONNEVILLE OR DIV HYDRAULIC L.. SEP 84

3/3

UNCLASSIFIED

TR-158 1

F/G 13/2

NL

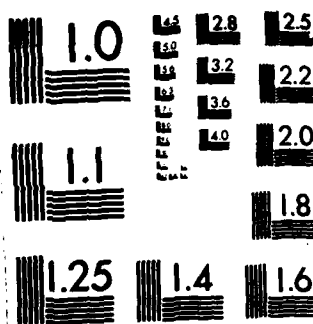
END

DATE

FILED

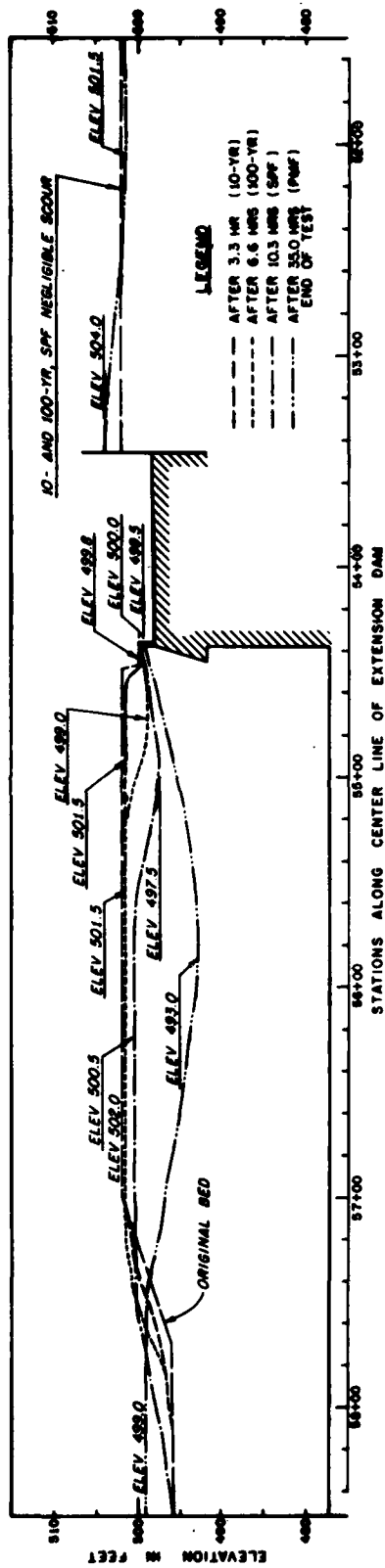
12 84

DTIC

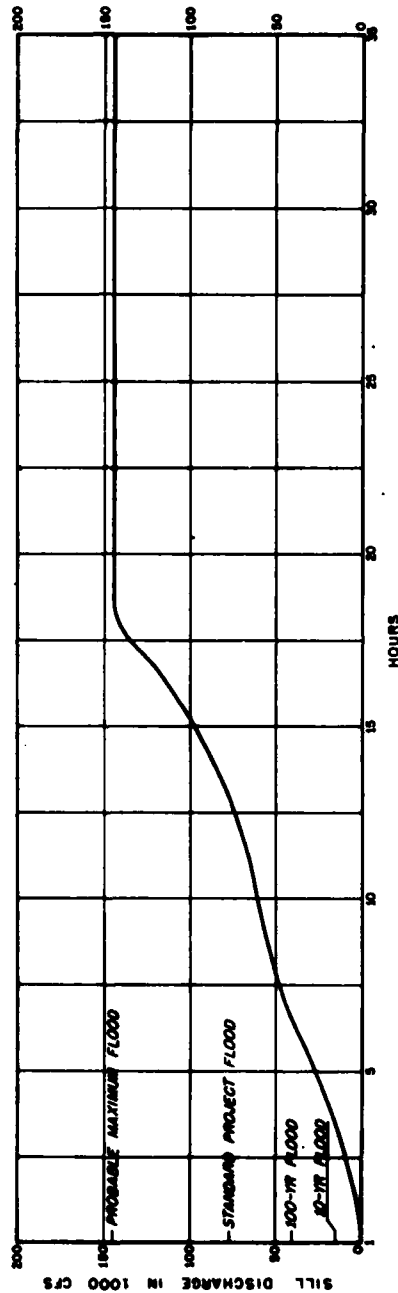


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

PLATE 54



AVERAGE SCOUR

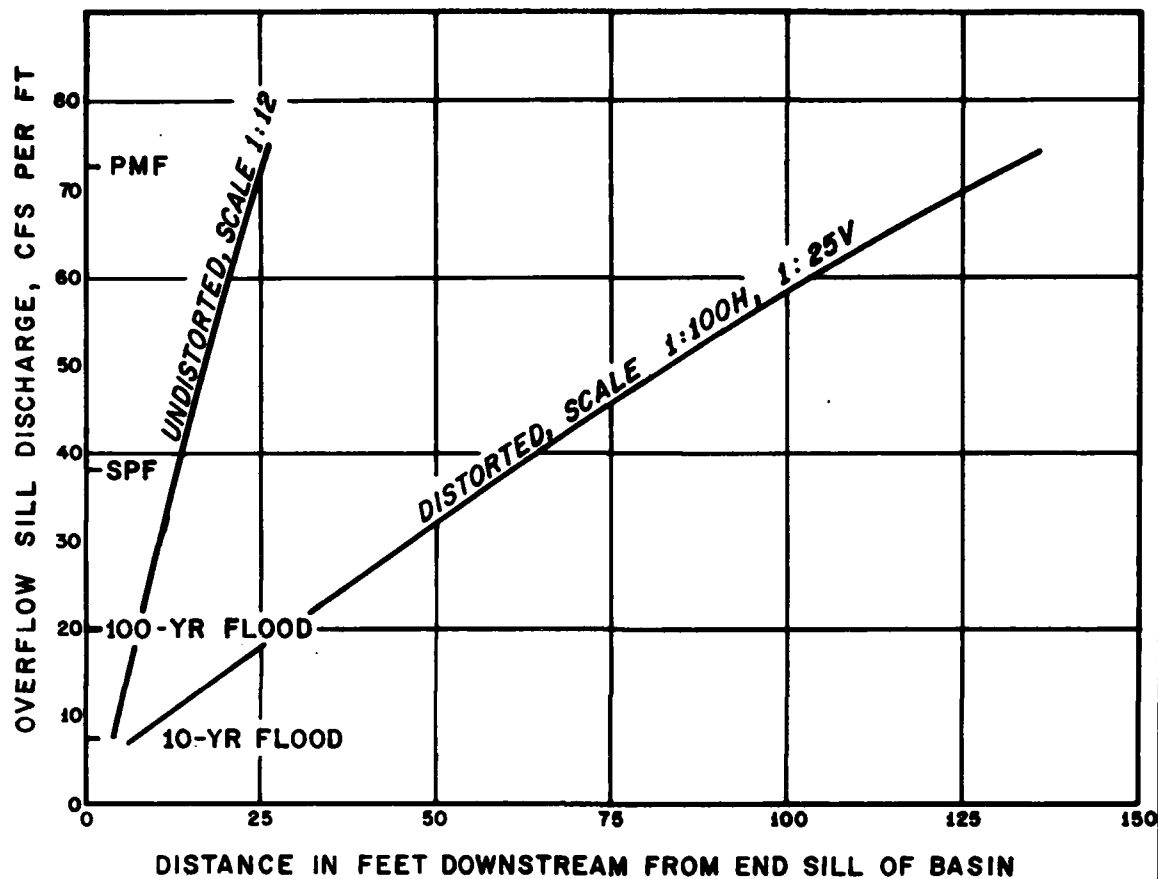


NOTE

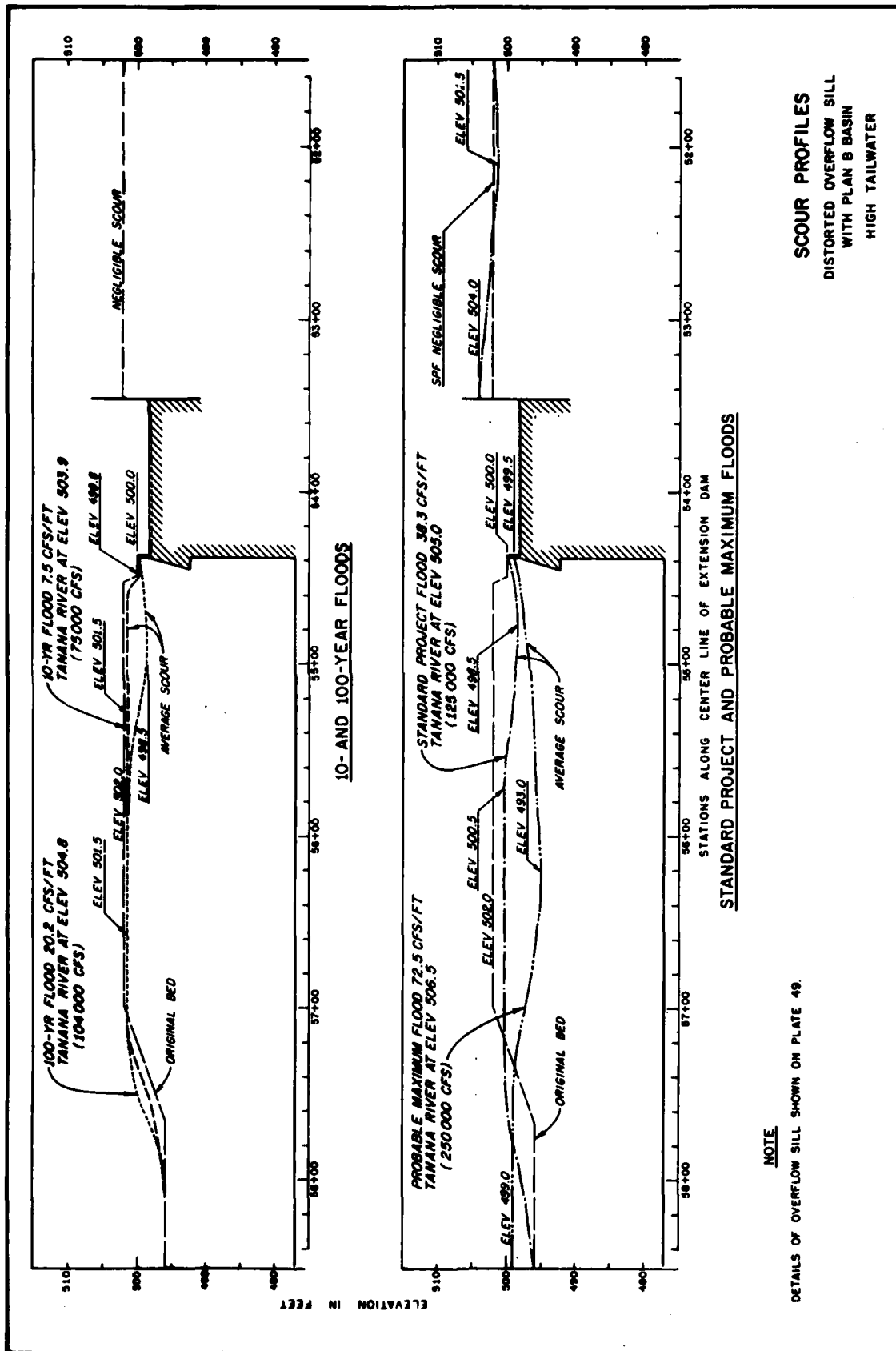
DETAILS OF OVERFLOW SILL SHOWN ON PLATE 48

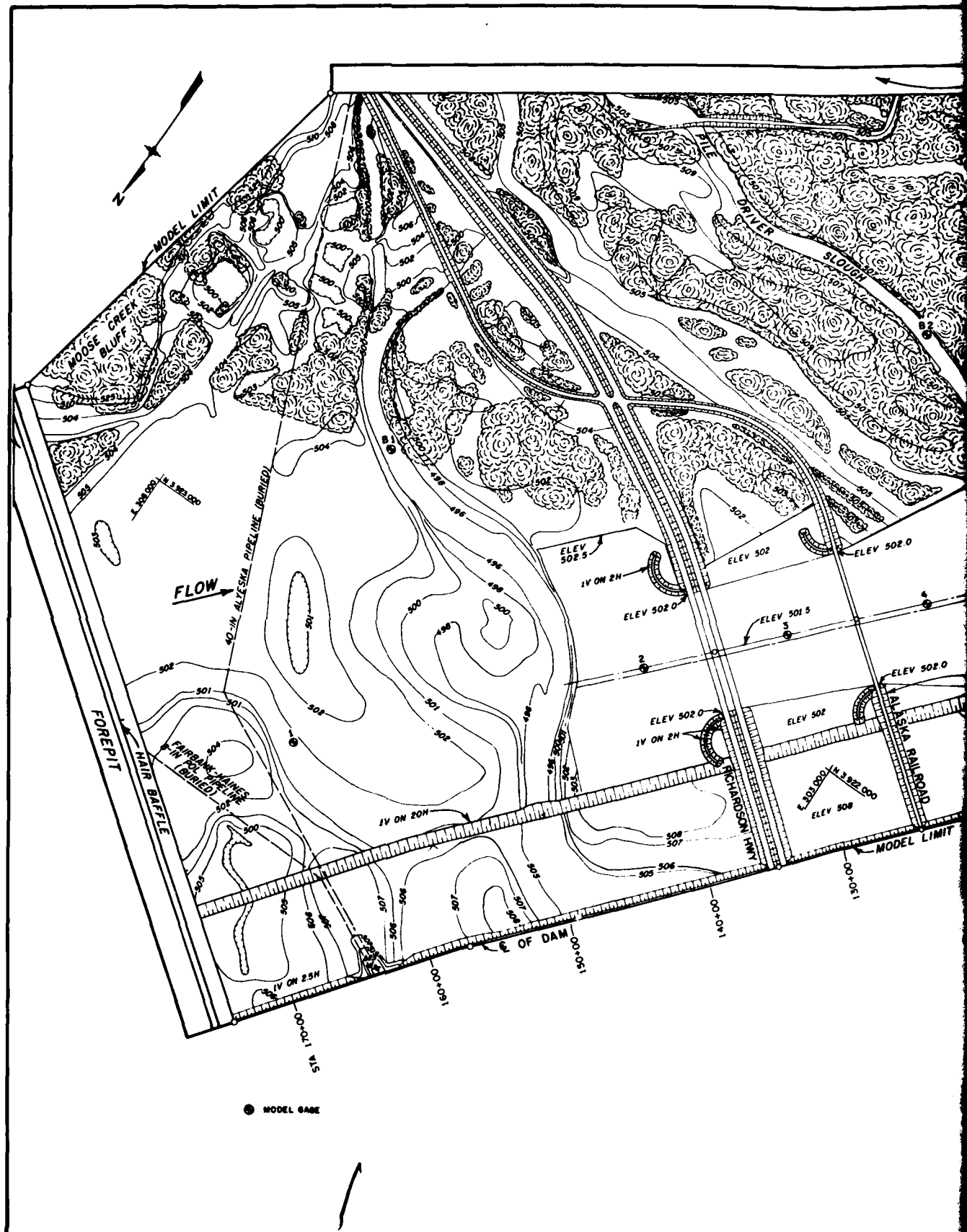
SCOUR PROFILES
DISTORTED OVERFLOW SILL
WITH PLAN B BASIN
LOW TAILWATER

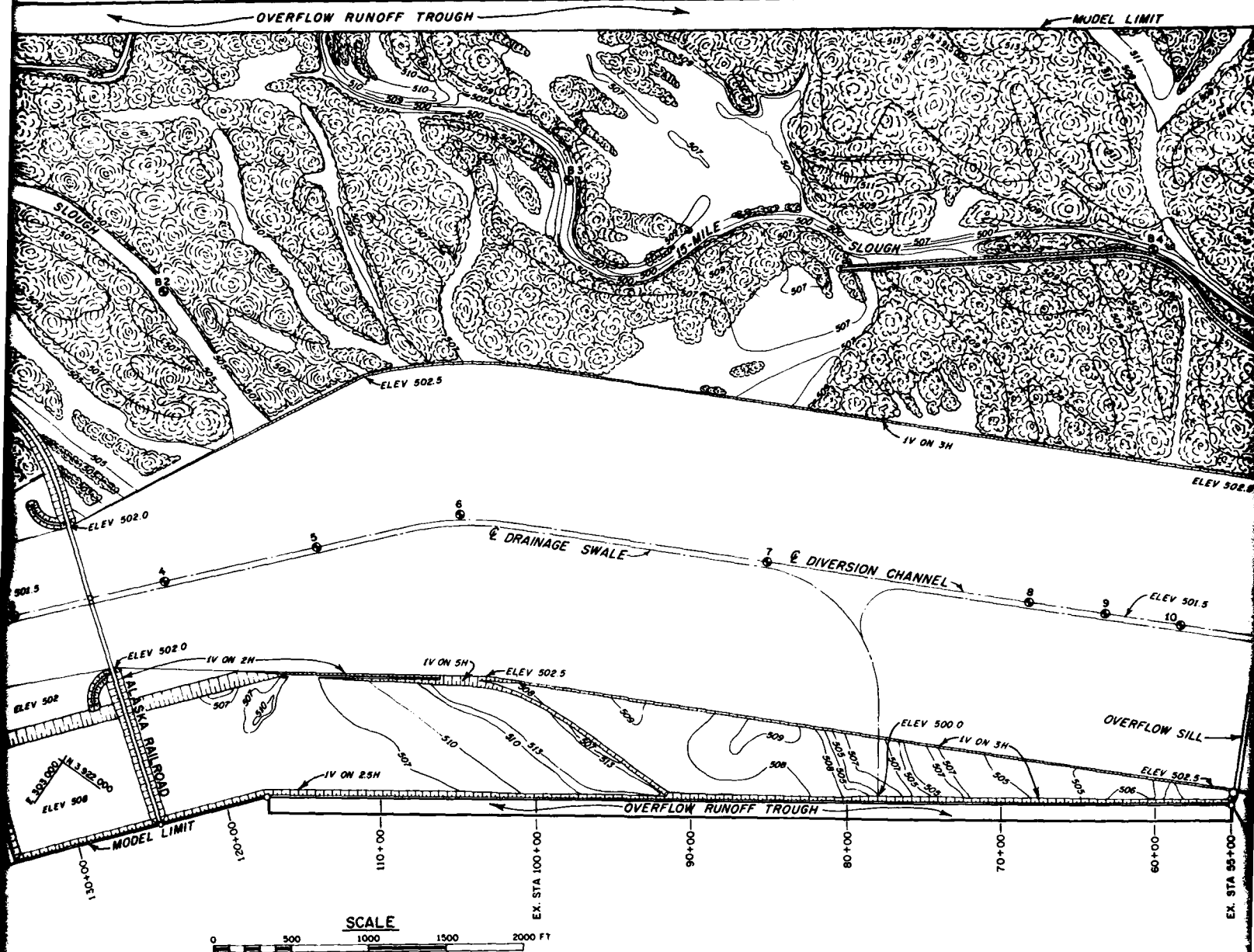
PROBABLE MAXIMUM FLOOD HYDROGRAPH



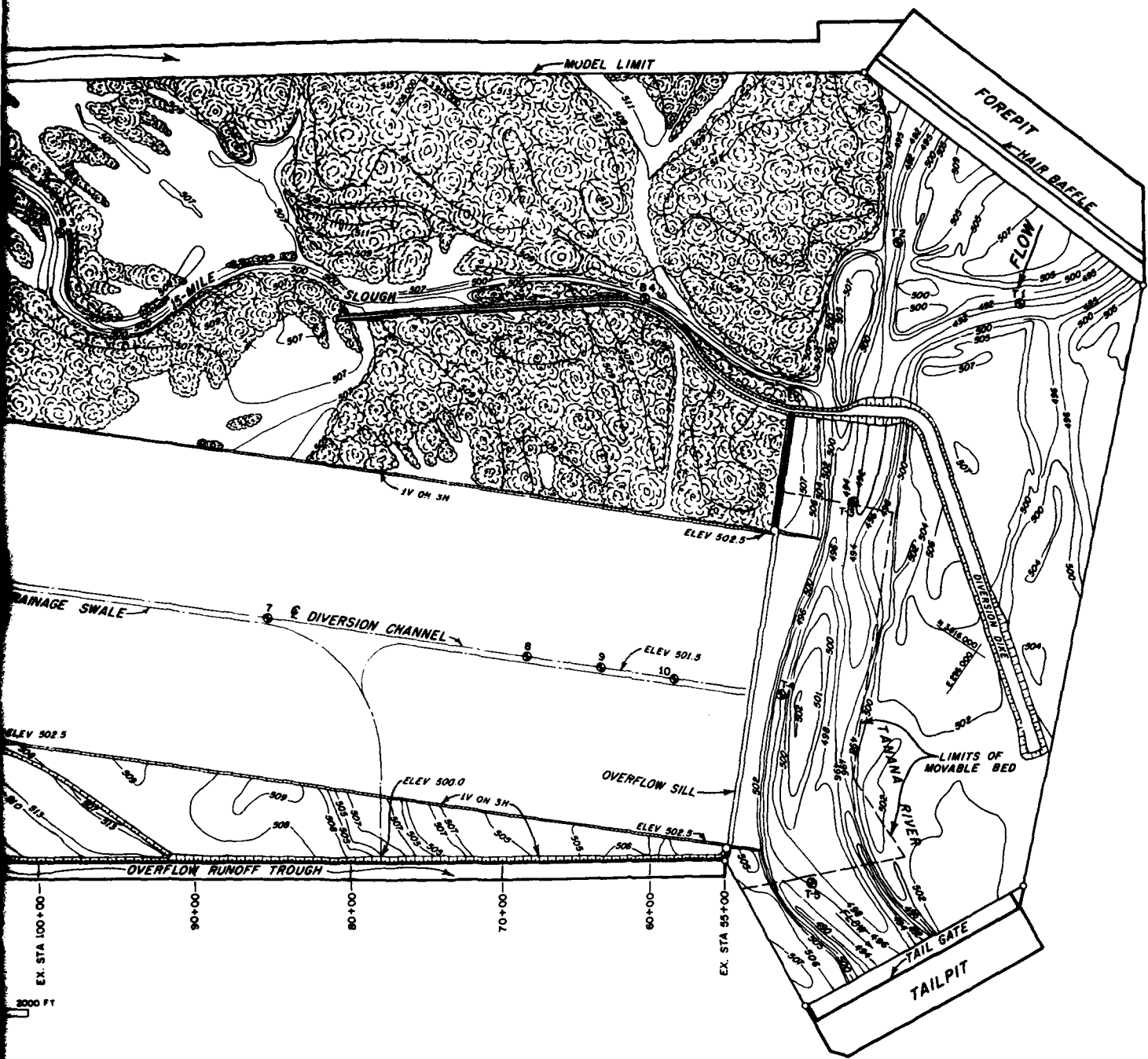
LOCATION OF MAXIMUM SCOUR
DOWNSTREAM FROM PLAN B BASIN
DISTORTED AND UNDISTORTED MODEL SCALES
LOW TAILWATER



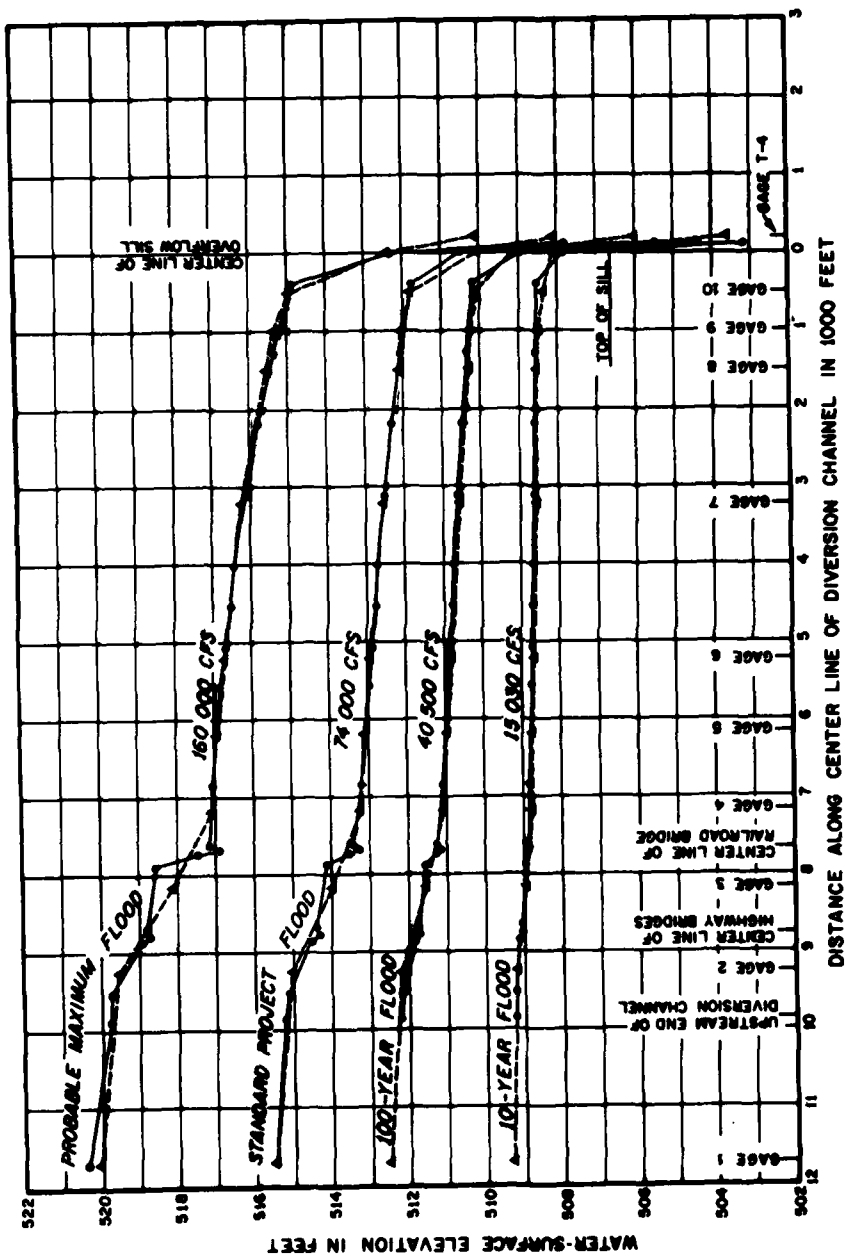




2

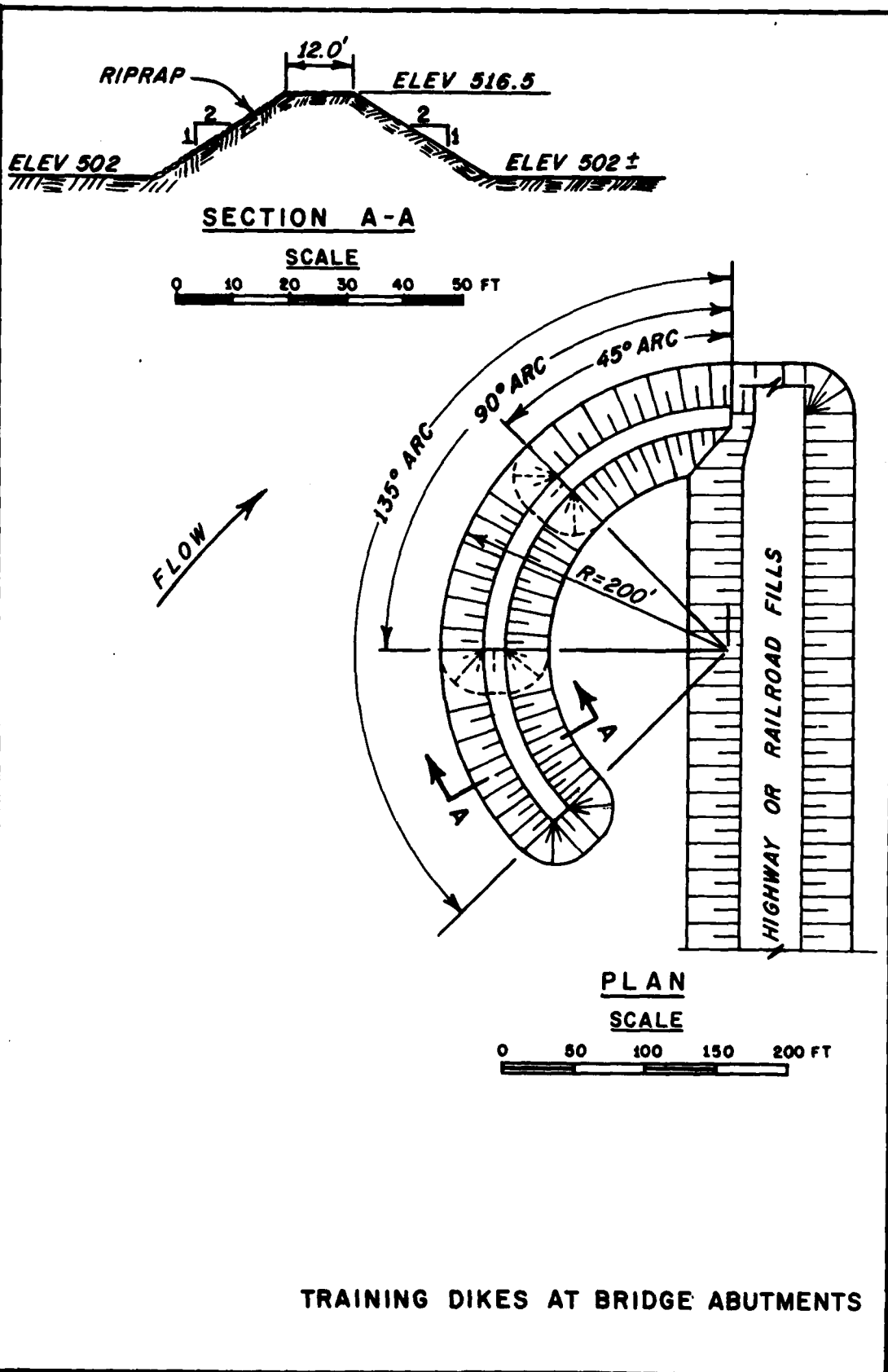


DIVERSION CHANNEL
FINAL DESIGN (PLAN B)

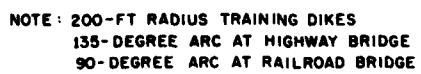


DIVERSION CHANNEL
WATER-SURFACE PROFILES

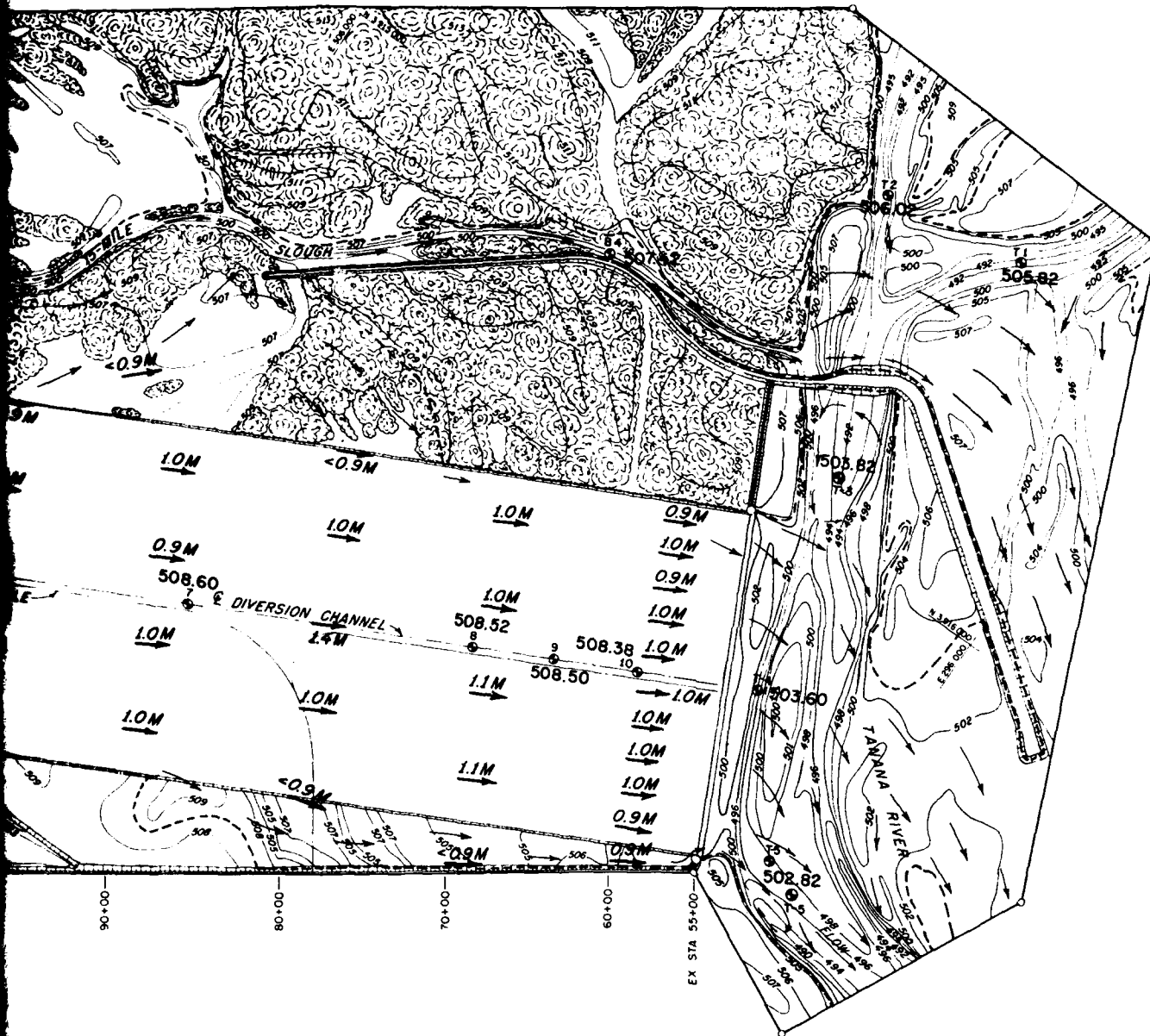
LEGEND
—●— COMPUTED
--- MODEL



TRAINING DIKES AT BRIDGE ABUTMENTS



**NOTE: 200-FT RADIUS TRAINING DIKES
135-DEGREE ARC AT HIGHWAY BRIDGE
90-DEGREE ARC AT RAILROAD BRIDGE**



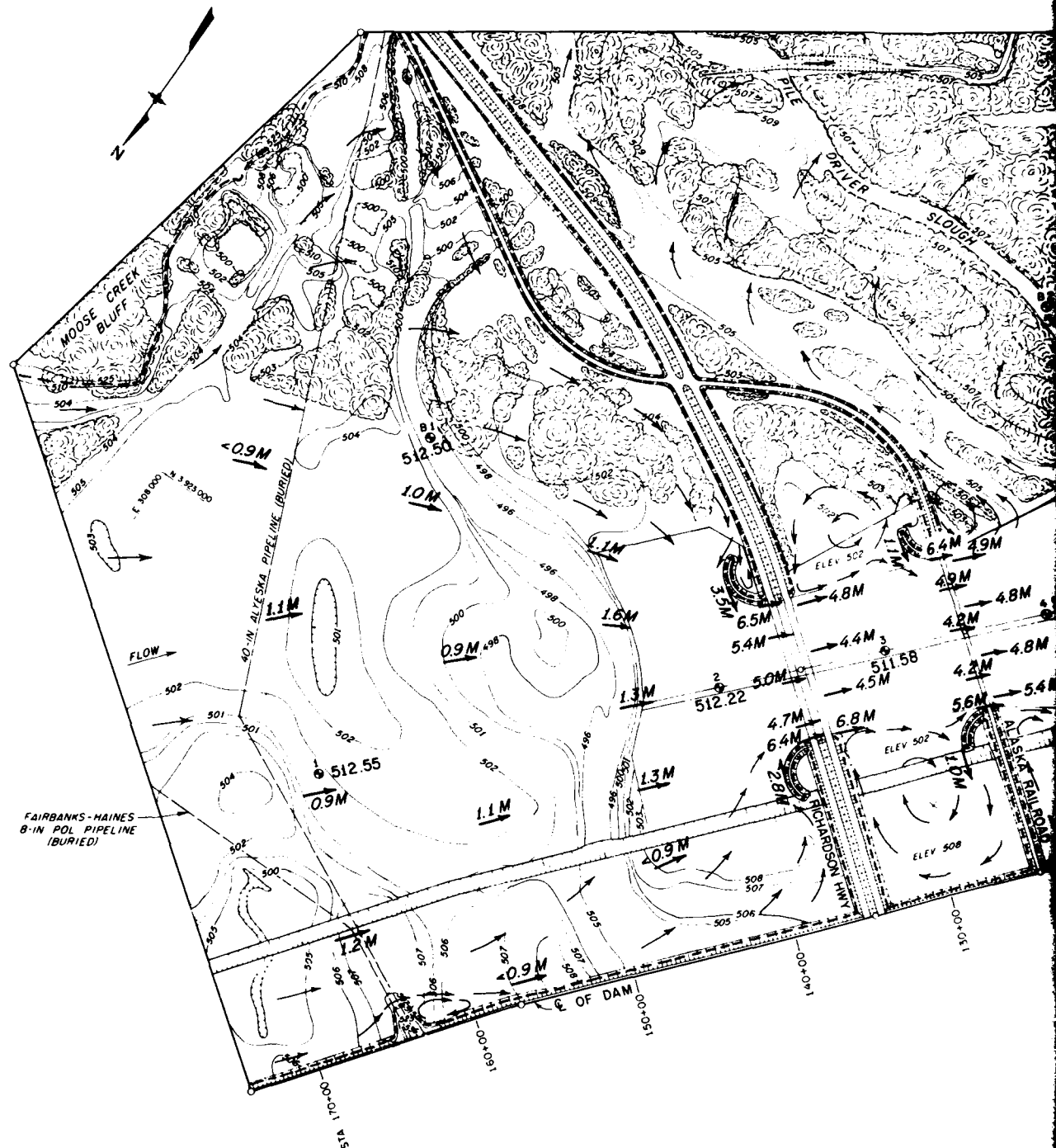
135- AND 90-DEGREE DIKS

10-YEAR FLOOD

**DIVERSION CHANNEL
FLOW CONDITIONS**

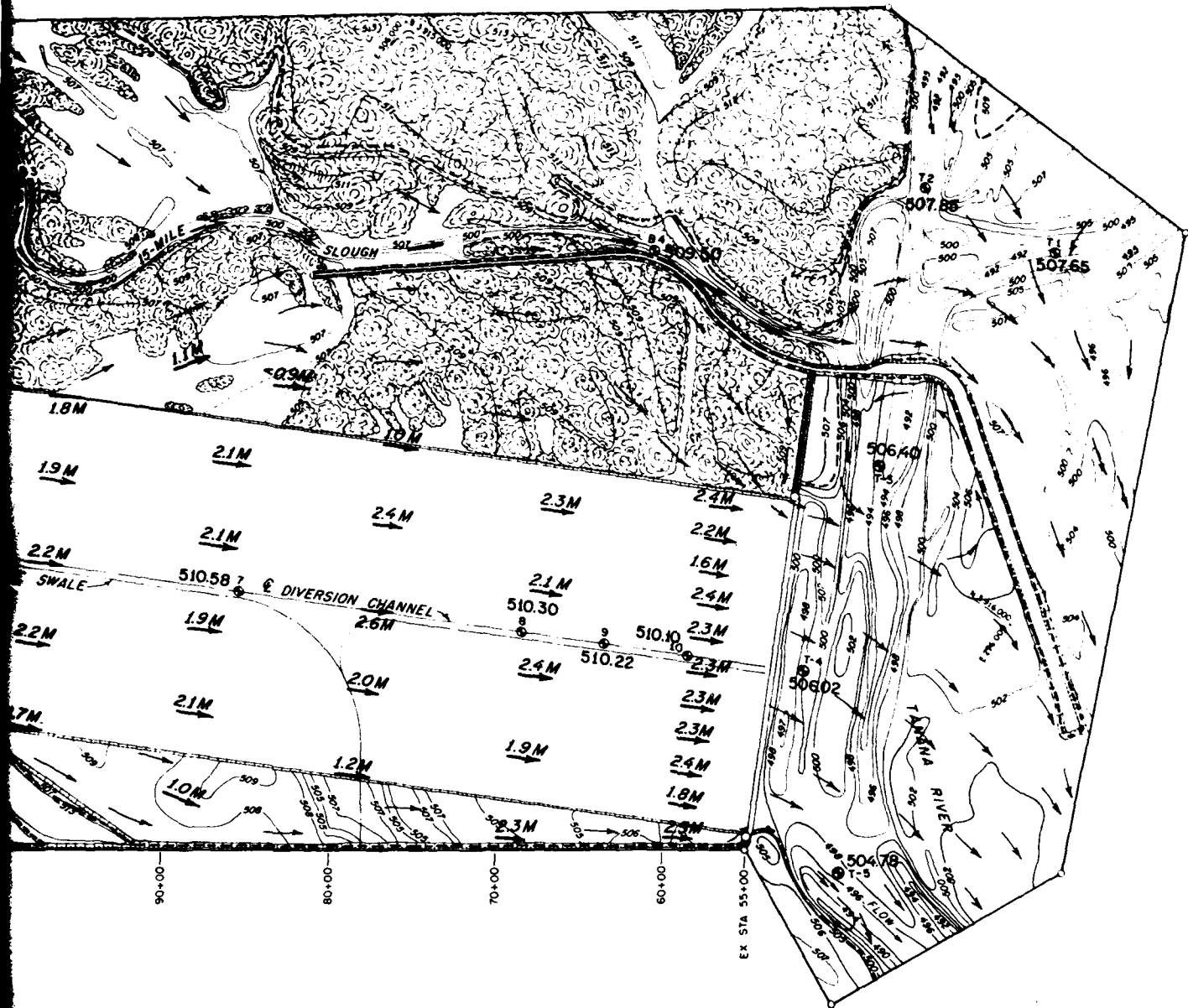
PLAN B

FLOODWAY DISCHARGE 15 030 CFS
TANANA RIVER DISCHARGE 75 000 CFS



LEGEND

- 3.1 VELOCITIES IN FPS
- T 3-FT DEPTH
- M MID-DEPTH
- B 3FT ABOVE BOTTOM
- 512.55 ● WATER-SURFACE ELEVATIONS



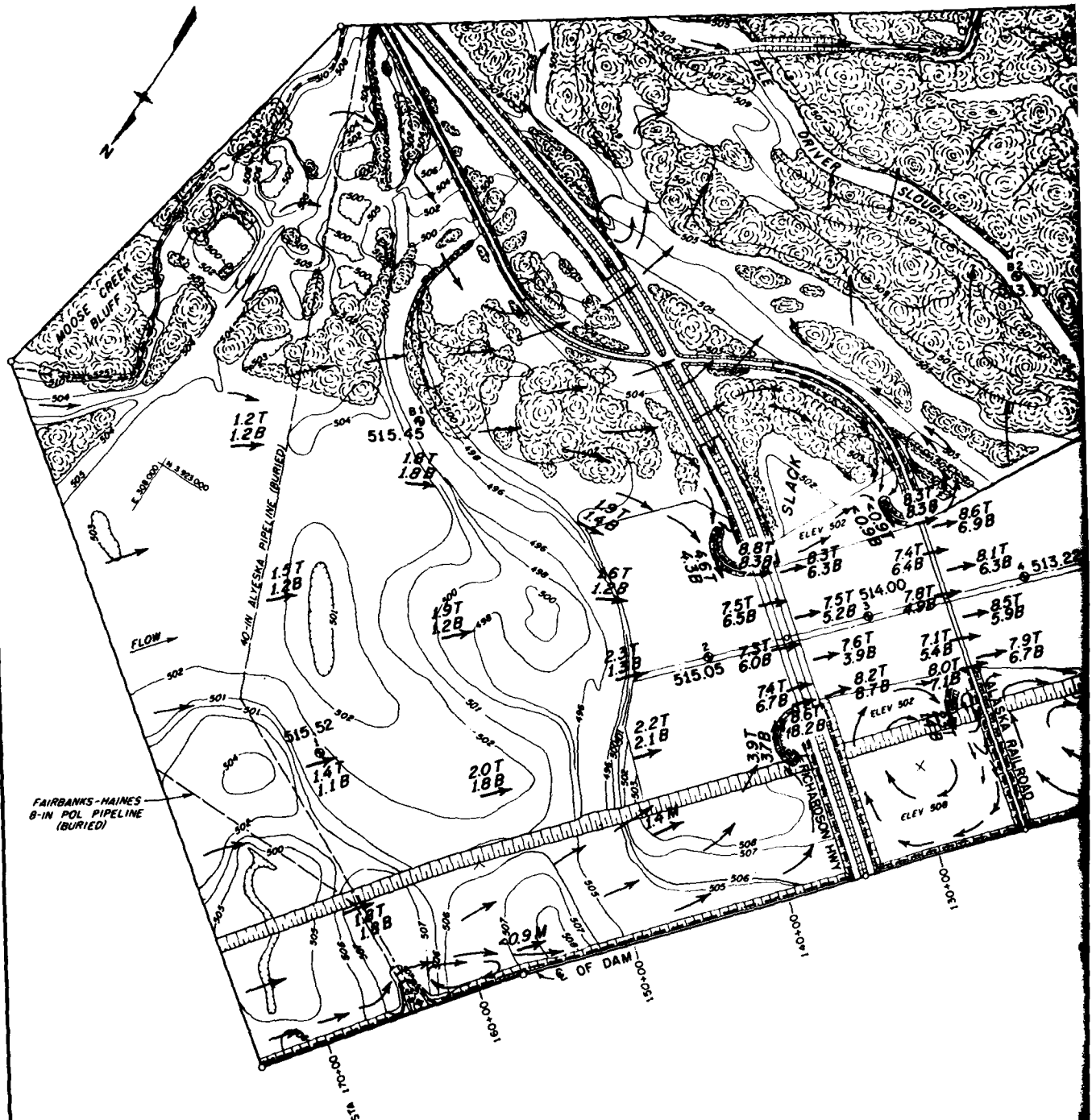
135- AND 90-DEGREE DIKS

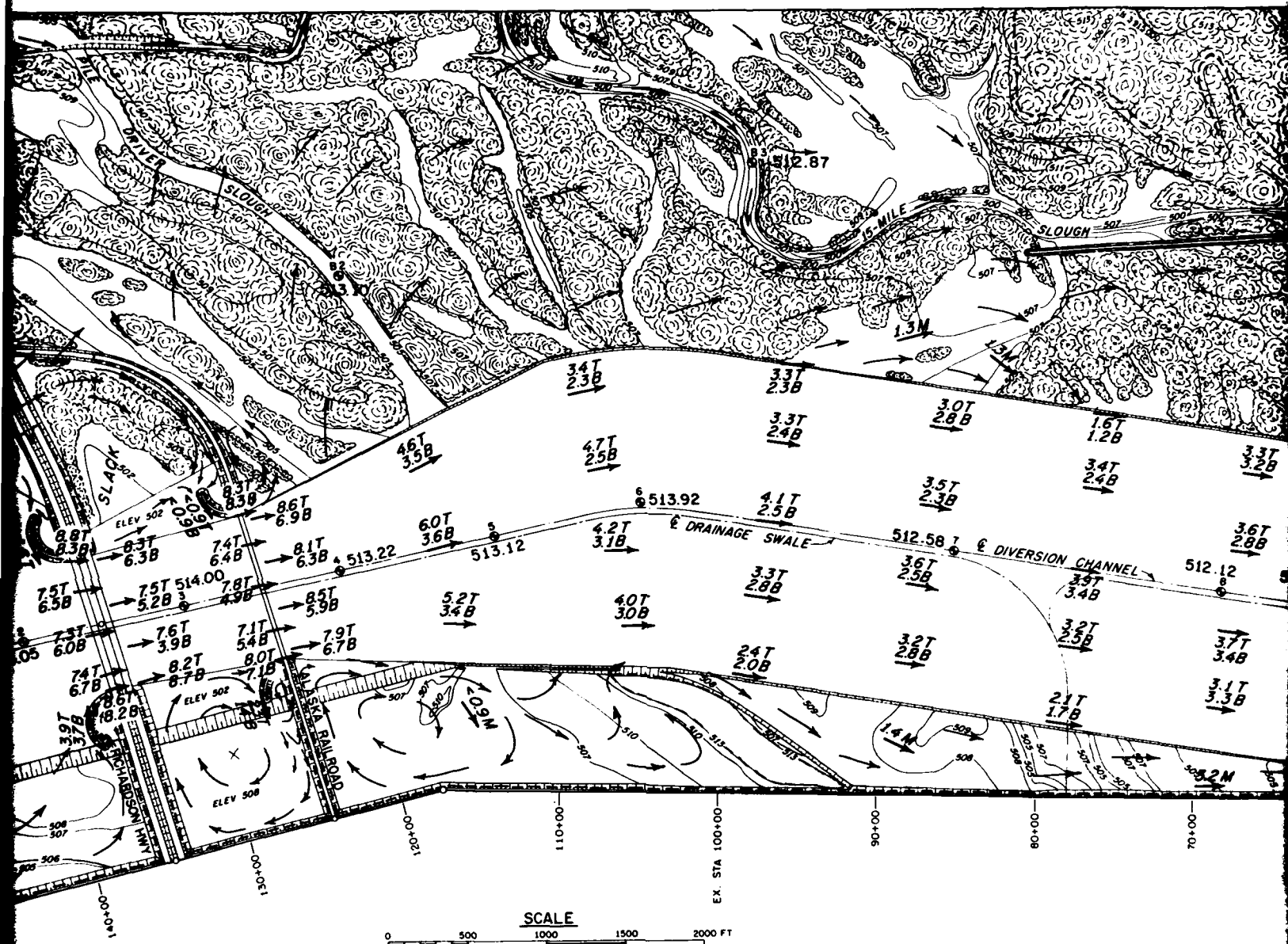
100-YEAR FLOOD

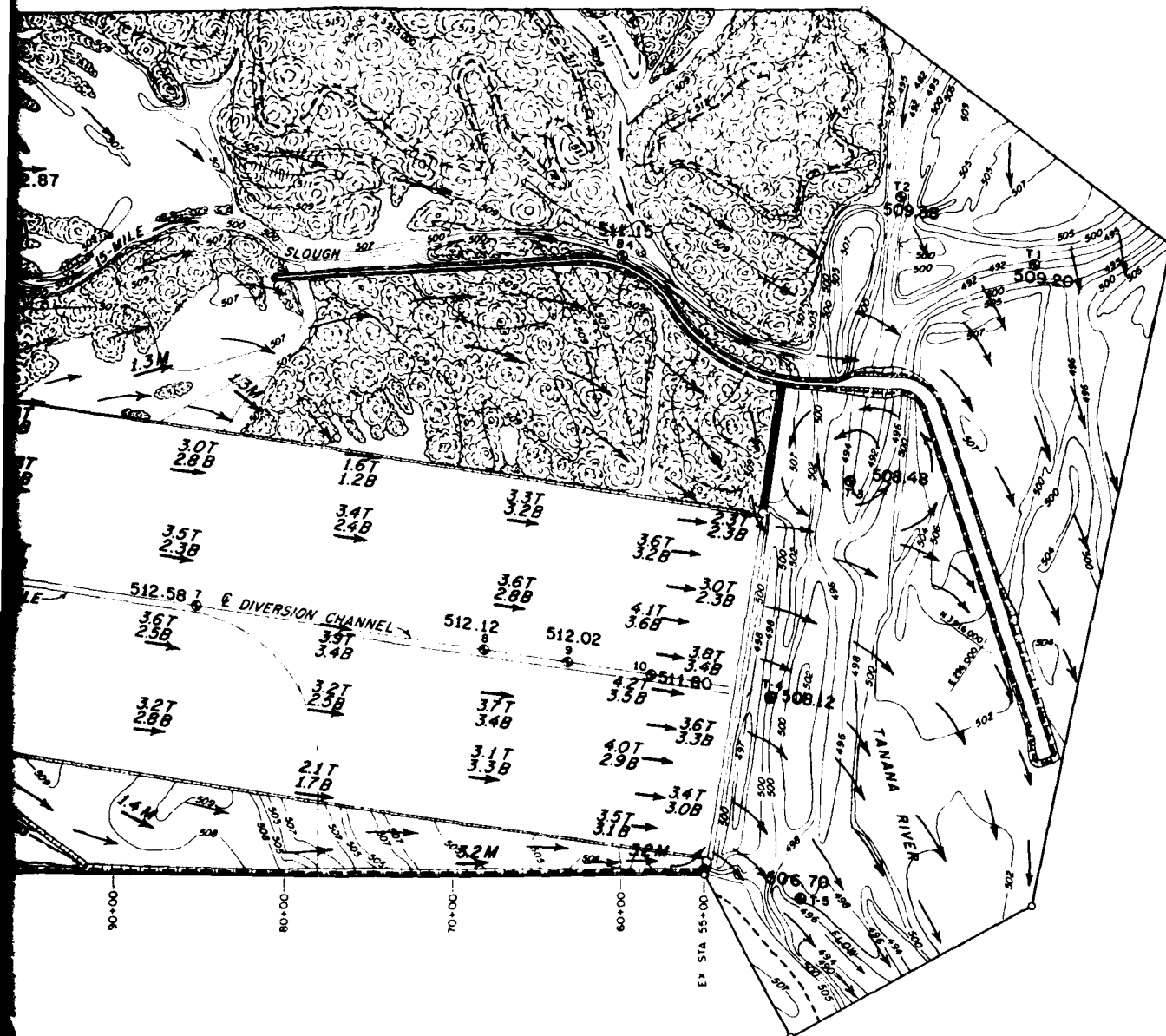
**DIVERSION CHANNEL
FLOW CONDITIONS**

PLAN B

FLOODWAY DISCHARGE 40 500 CFS
TANANA RIVER DISCHARGE 104 000 CFS





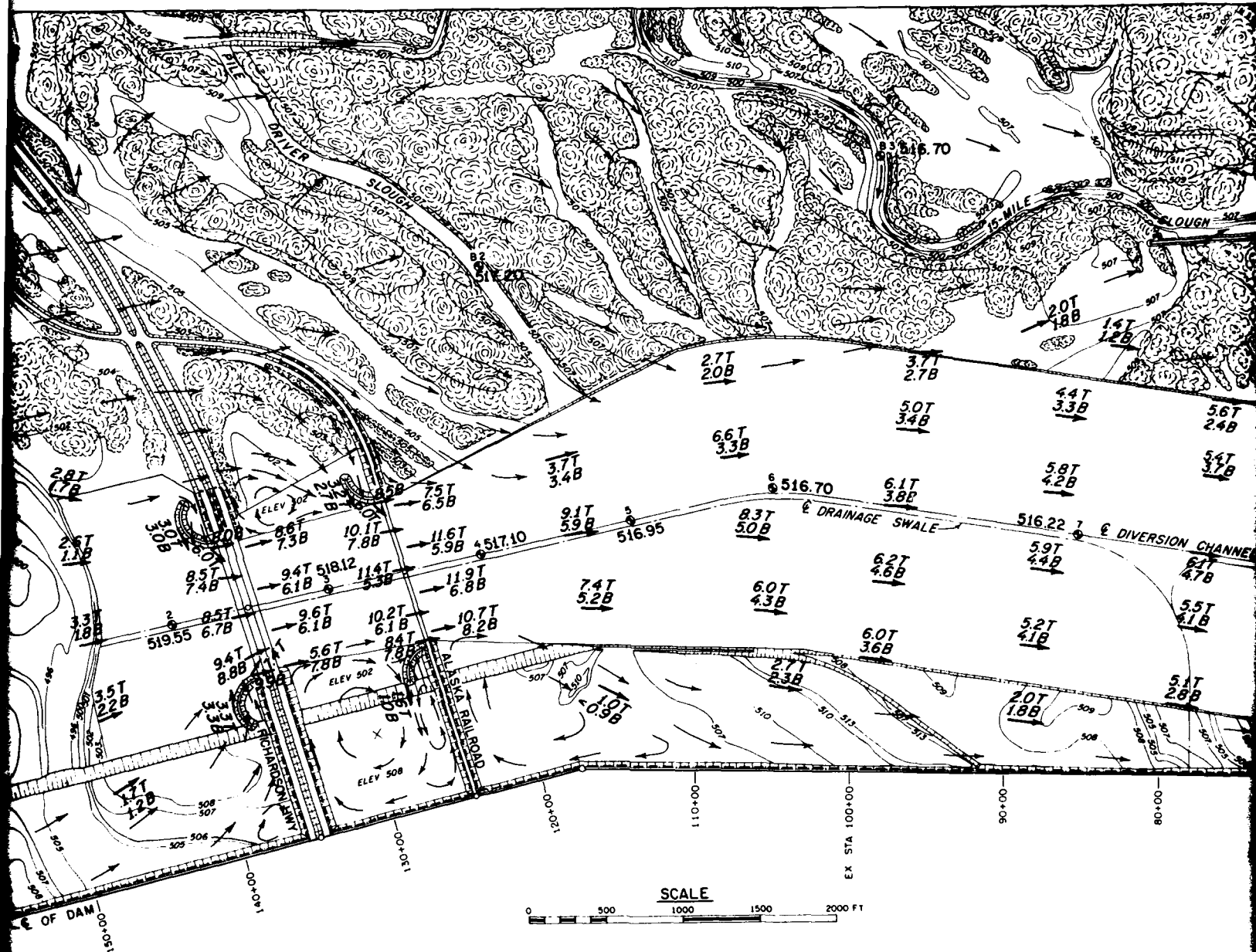


135- AND 90-DEGREE DIKS
STANDARD PROJECT FLOOD

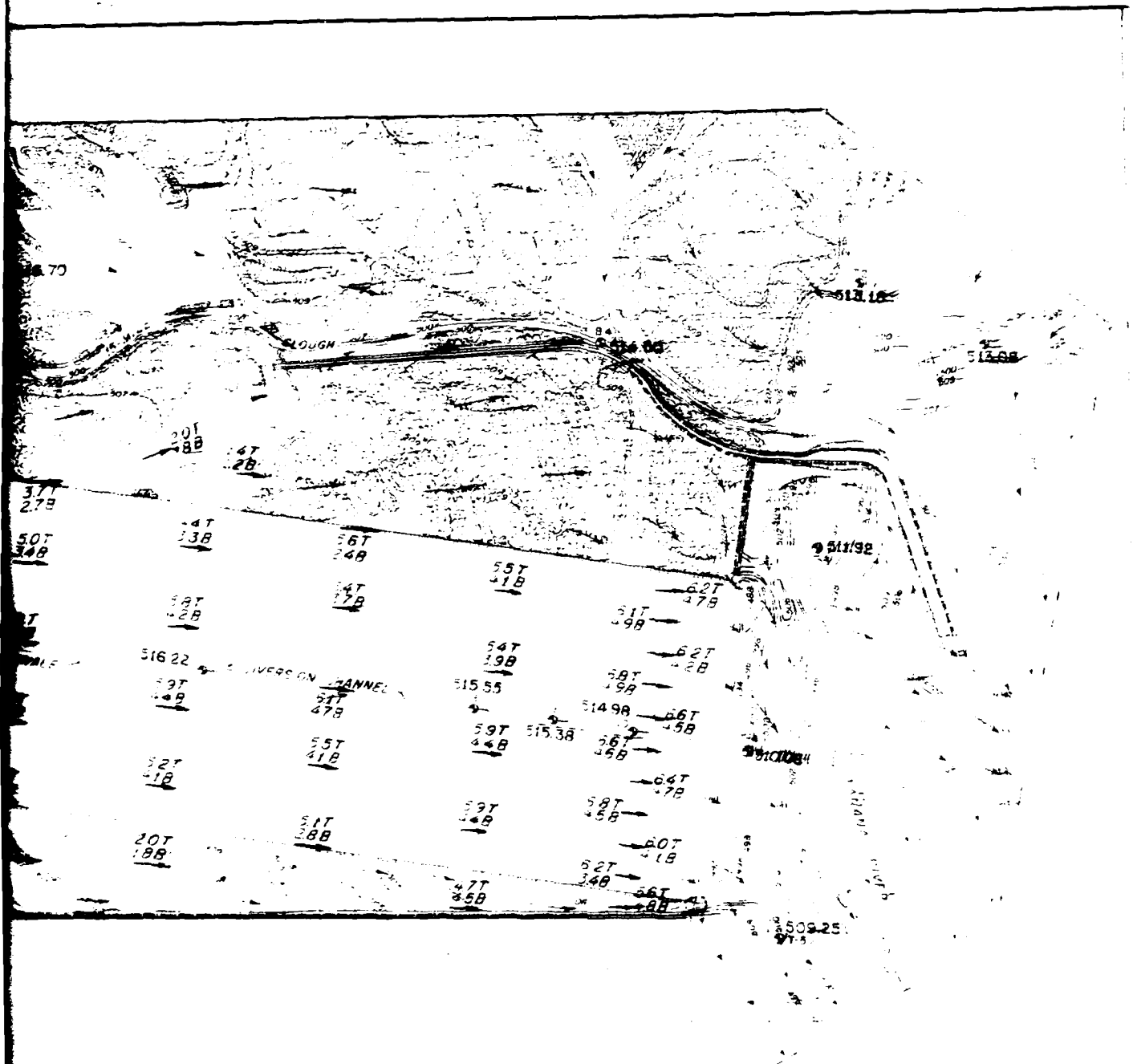
**DIVERSION CHANNEL
 FLOW CONDITIONS**

PLAN B

FLOODWAY DISCHARGE 74 000 CFS
 TANANA RIVER DISCHARGE 125 000 CFS



NOTE: 200-FT RADIUS TRAINING DIKS
135-DEGREE ARC AT HIGHWAY BRIDGE
90-DEGREE ARC AT RAILROAD BRIDGE



55- AND 90-DEGREE DIKES
PROBABLE MAXIMUM FLOOD

DIVERSION CHANNEL
FLOW CONDITIONS

PLAN 3

FLOODWAY DISCHARGE 160,000 CFS
CANANA RIVER DISCHARGE 150,000 CFS

PLATE 53

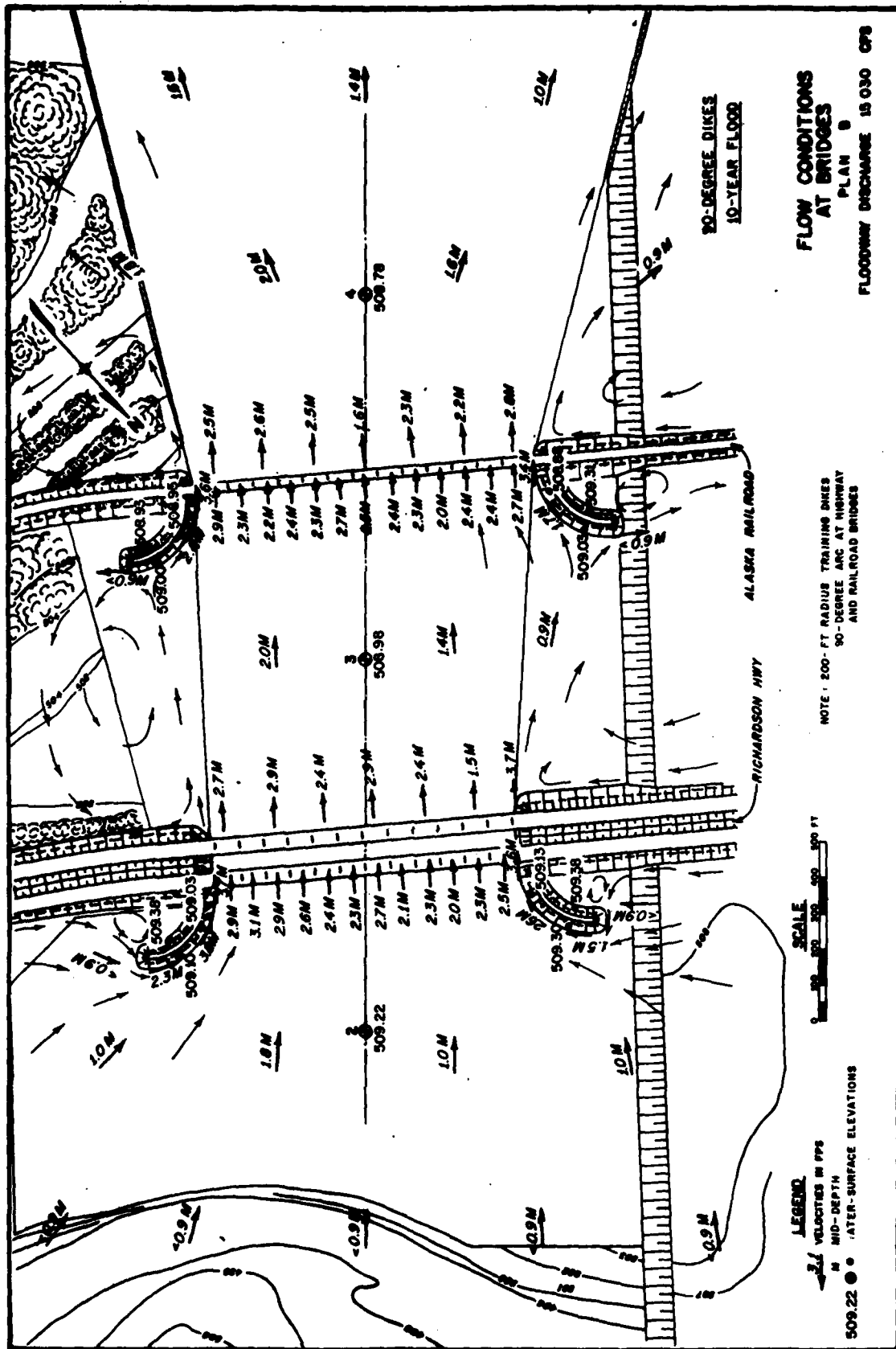


PLATE 64

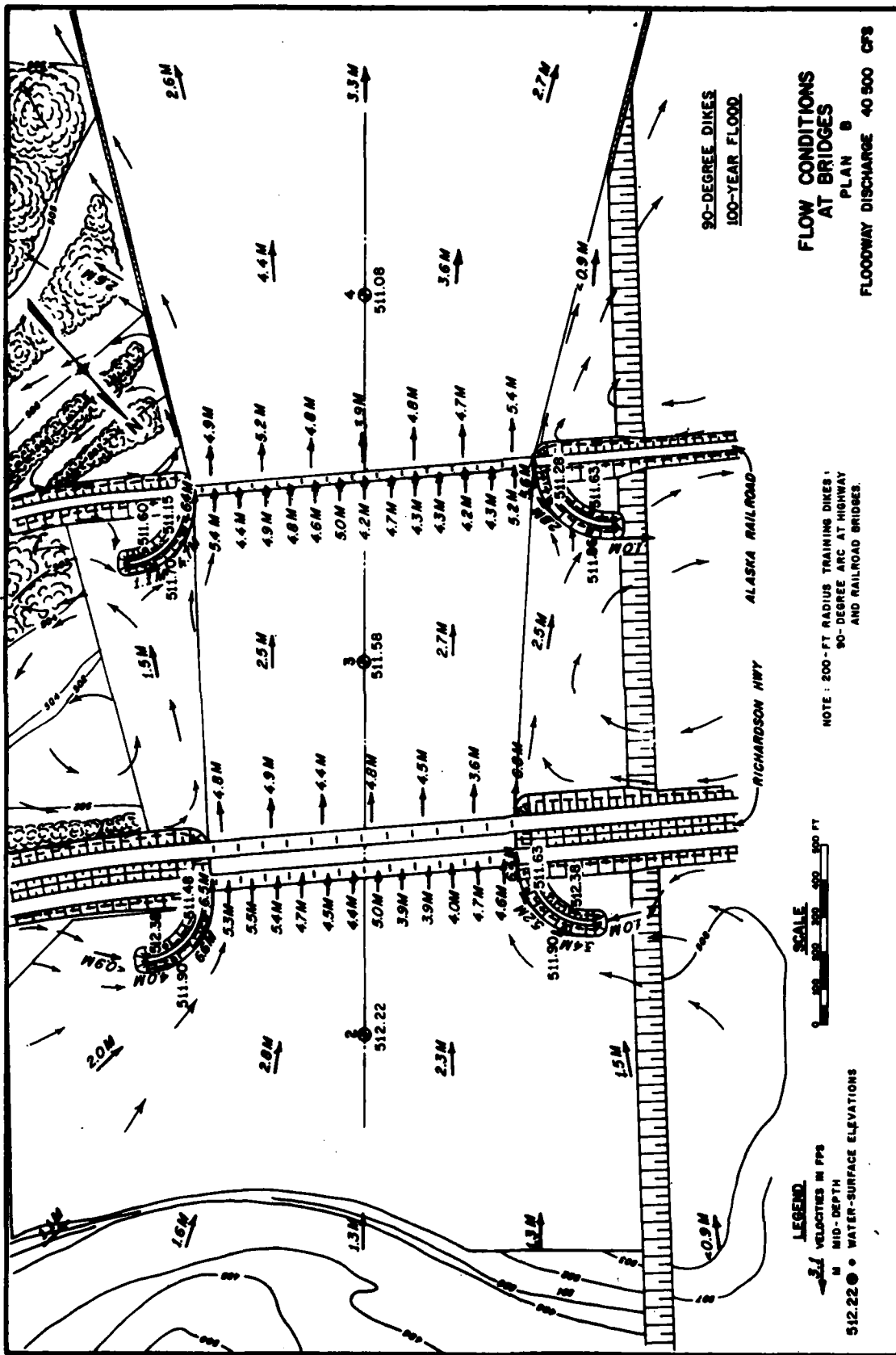
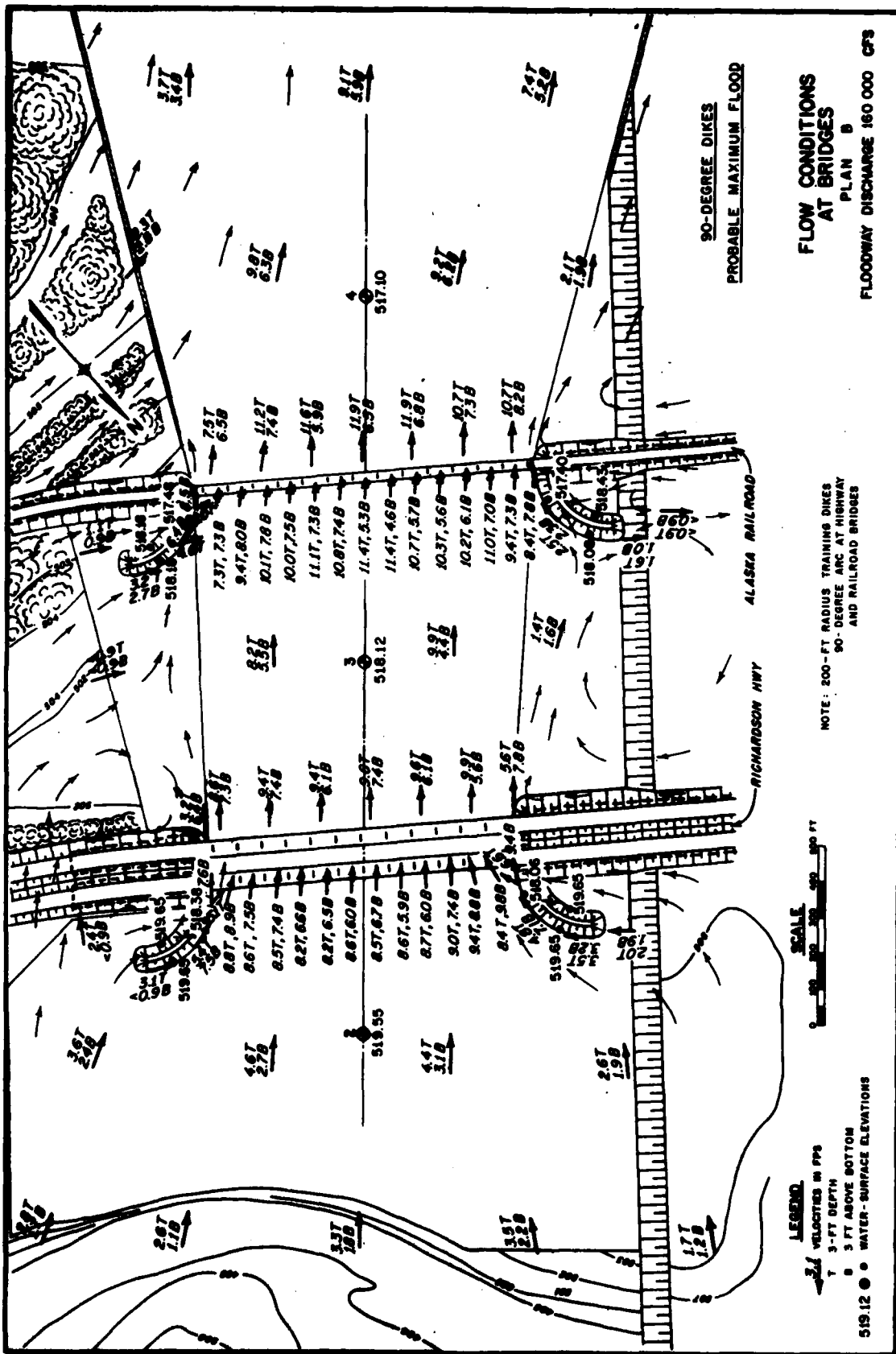
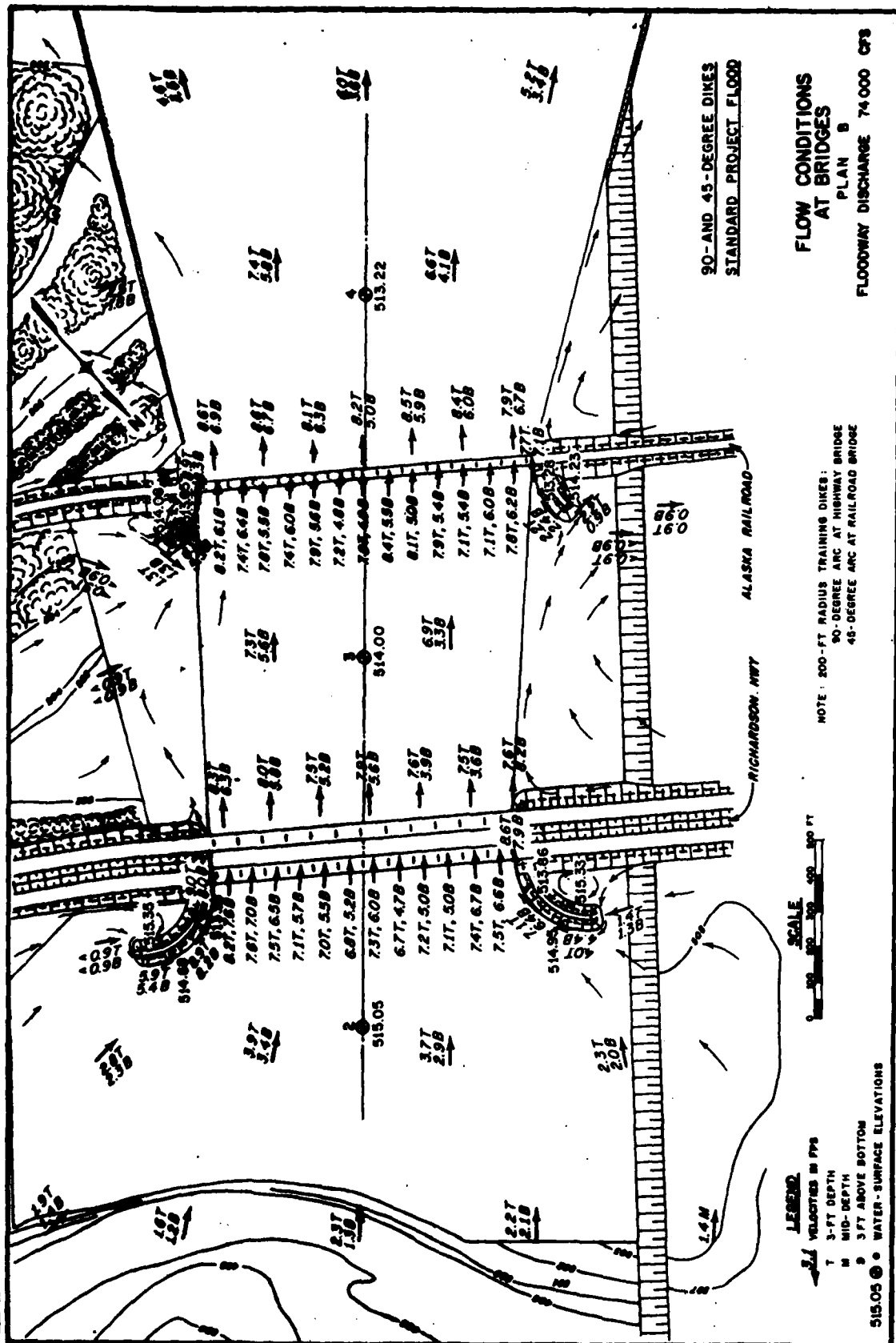


PLATE 65



12



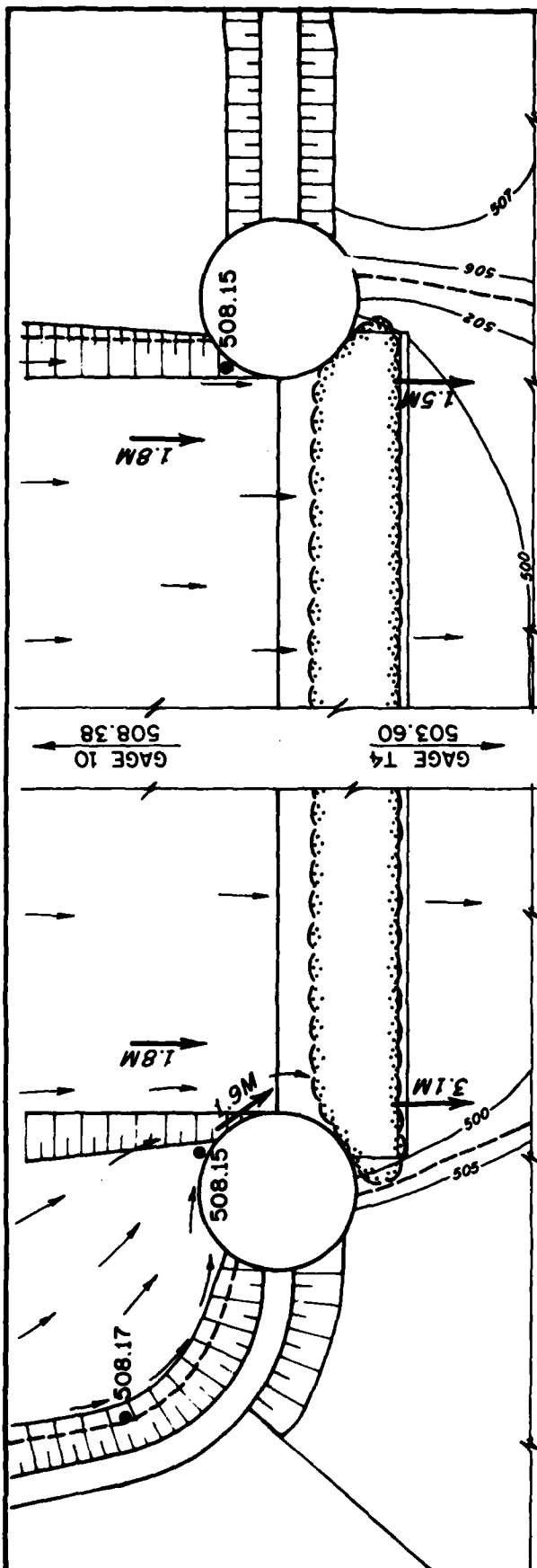


PLATE 71

10-YEAR FLOOD

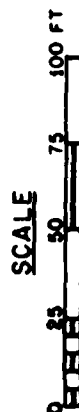
**DIVERSION CHANNEL
FLOW CONDITIONS AND SCOUR
AT OVERFLOW SILL**
PLAN B
FLOODWAY DISCHARGE 15030 CFS

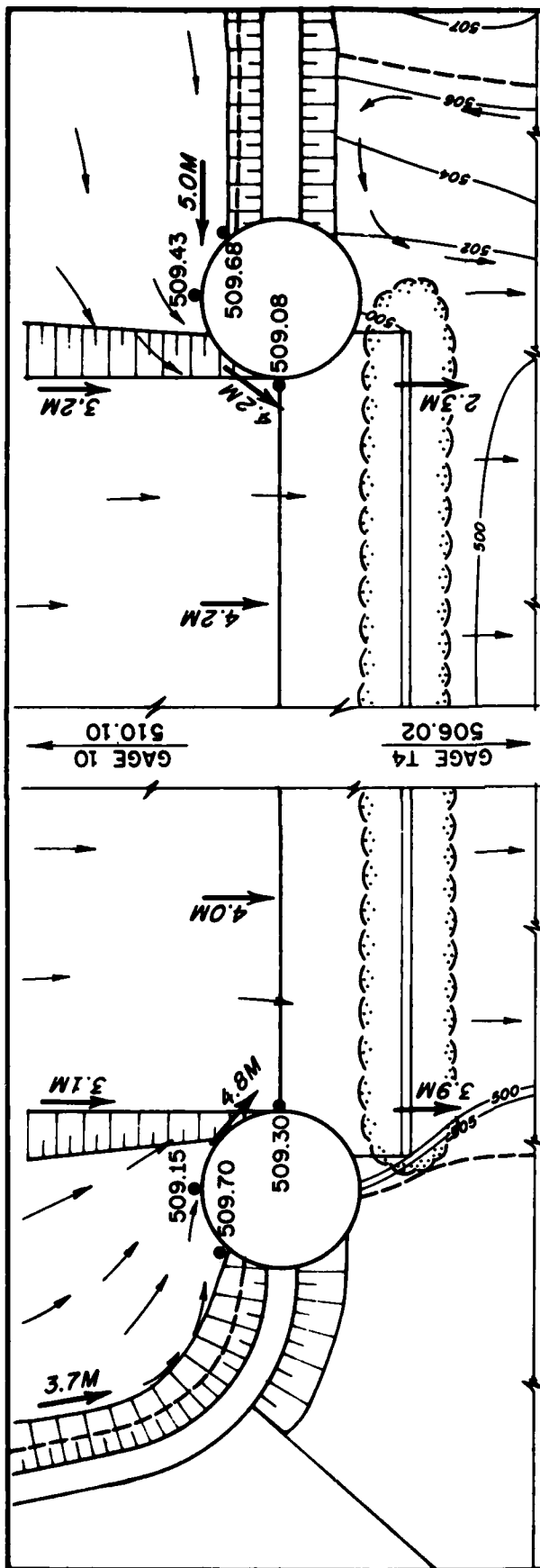
NOTE

DETAILS OF OVERFLOW SILL AND
BASIN SHOWN ON PLATE 49.

LEGEND

1.8 VELOCITIES IN FPS
M MID-DEPTH
508.15 WATER-SURFACE ELEVATIONS
BOIL OUTLINE





100-YEAR FLOOD

DIVERSION CHANNEL
FLOW CONDITIONS AND SCOUR
AT OVERFLOW SILL

PLAN B

FLOODWAY DISCHARGE 40 500 CFS

NOTE

DETAILS OF OVERFLOW SILL AND
BASIN SHOWN ON PLATE 49.

LEGEND

3.1 VELOCITIES IN FPS
M MID-DEPTH

509.30 • WATER-SURFACE ELEVATIONS

--- BOIL OUTLINE



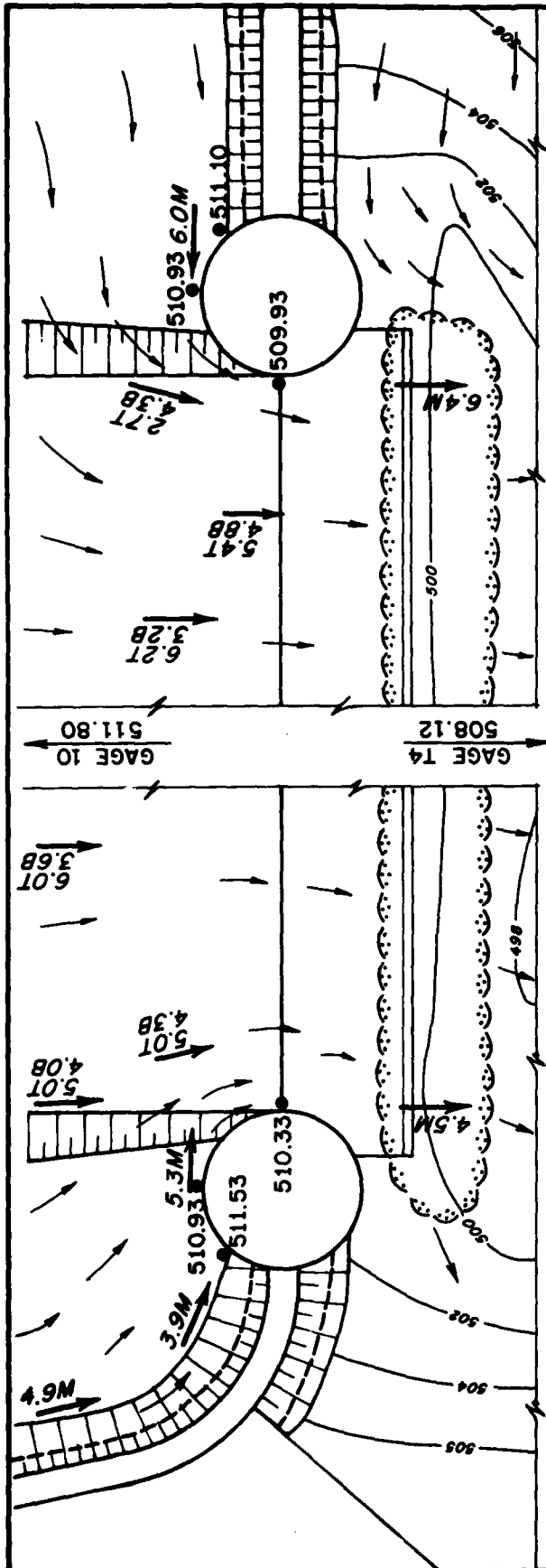


PLATE 73

STANDARD PROJECT FLOOD

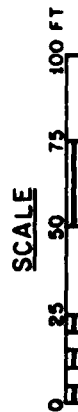
DIVERSION CHANNEL
FLOW CONDITIONS AND SCOUR
AT OVERFLOW SILL
PLAN B
FLOODWAY DISCHARGE 74000 CFS

NOTE

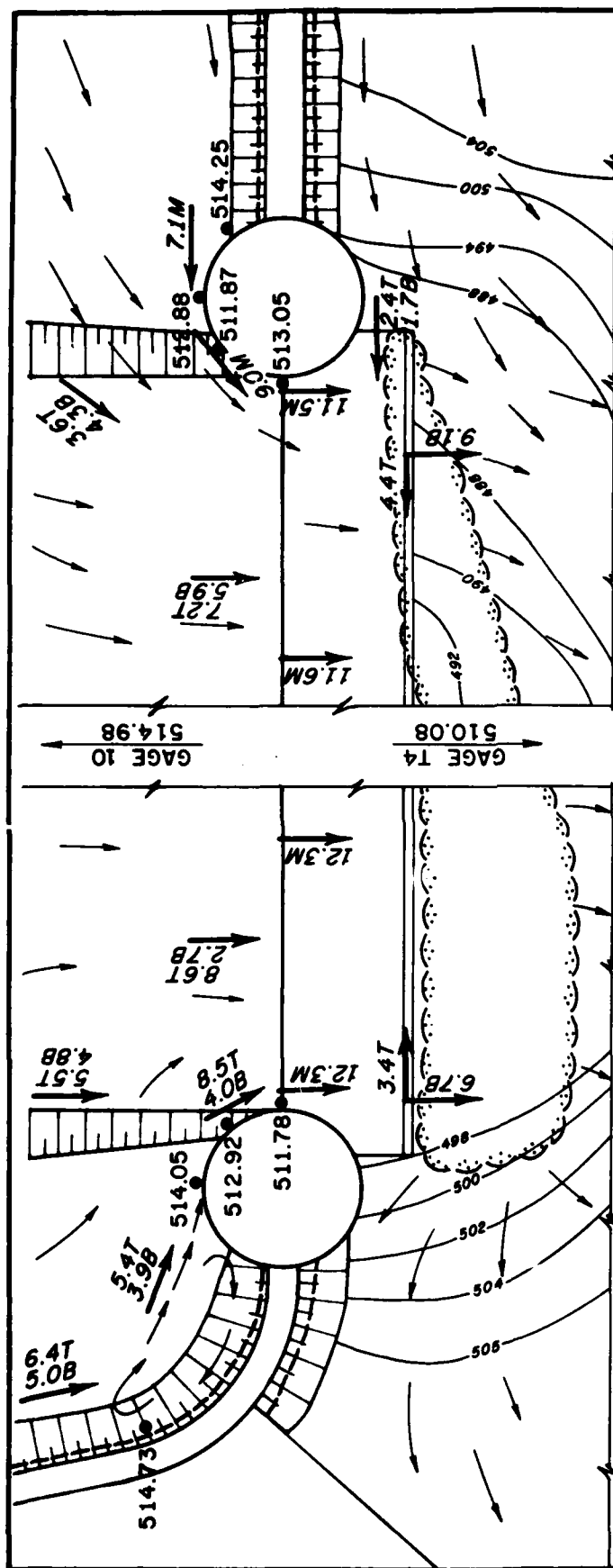
DETAILS OF OVERFLOW SILL AND
BASIN SHOWN ON PLATE 49.

LEGEND

5.0 VELOCITIES IN FPS
T 3-FT DEPTH
M MID-DEPTH
B 3 FT ABOVE BOTTOM
511.53 WATER SURFACE ELEVATIONS
BOIL OUTLINE



2



PROBABLE MAXIMUM FLOOD

**DIVERSION CHANNEL
FLOW CONDITIONS AND SCOUR
AT OVERFLOW SILL**

PLAN B

FLOODWAY DISCHARGE 160000 CFS

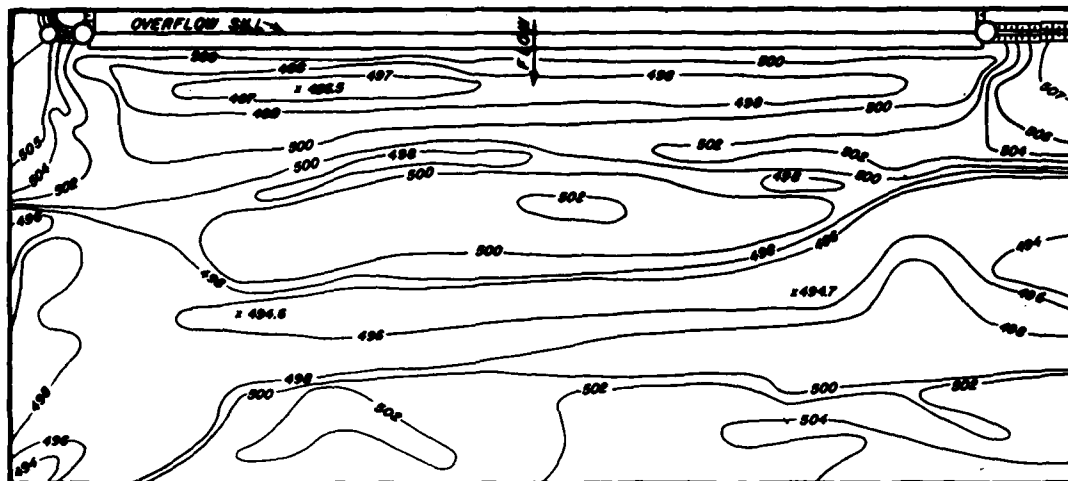
NOTE

DETAILS OF OVERFLOW SILL AND
BASIN SHOWN ON PLATE 49.

LEGEND

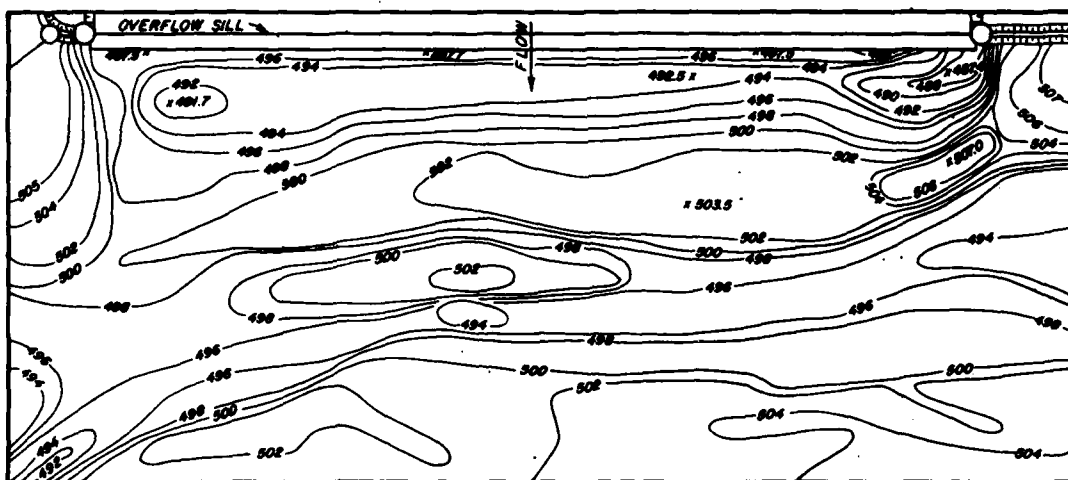
- 5.8 VELOCITIES IN FPS
- T 3-FT DEPTH
- M MID-DEPTH
- B 3 FT ABOVE BOTTOM
- 514.05 WATER-SURFACE ELEVATIONS
- BOIL OUTLINE





STANDARD PROJECT FLOOD

74 000 CFS



PROBABLE MAXIMUM FLOOD

160 000 CFS

NOTES

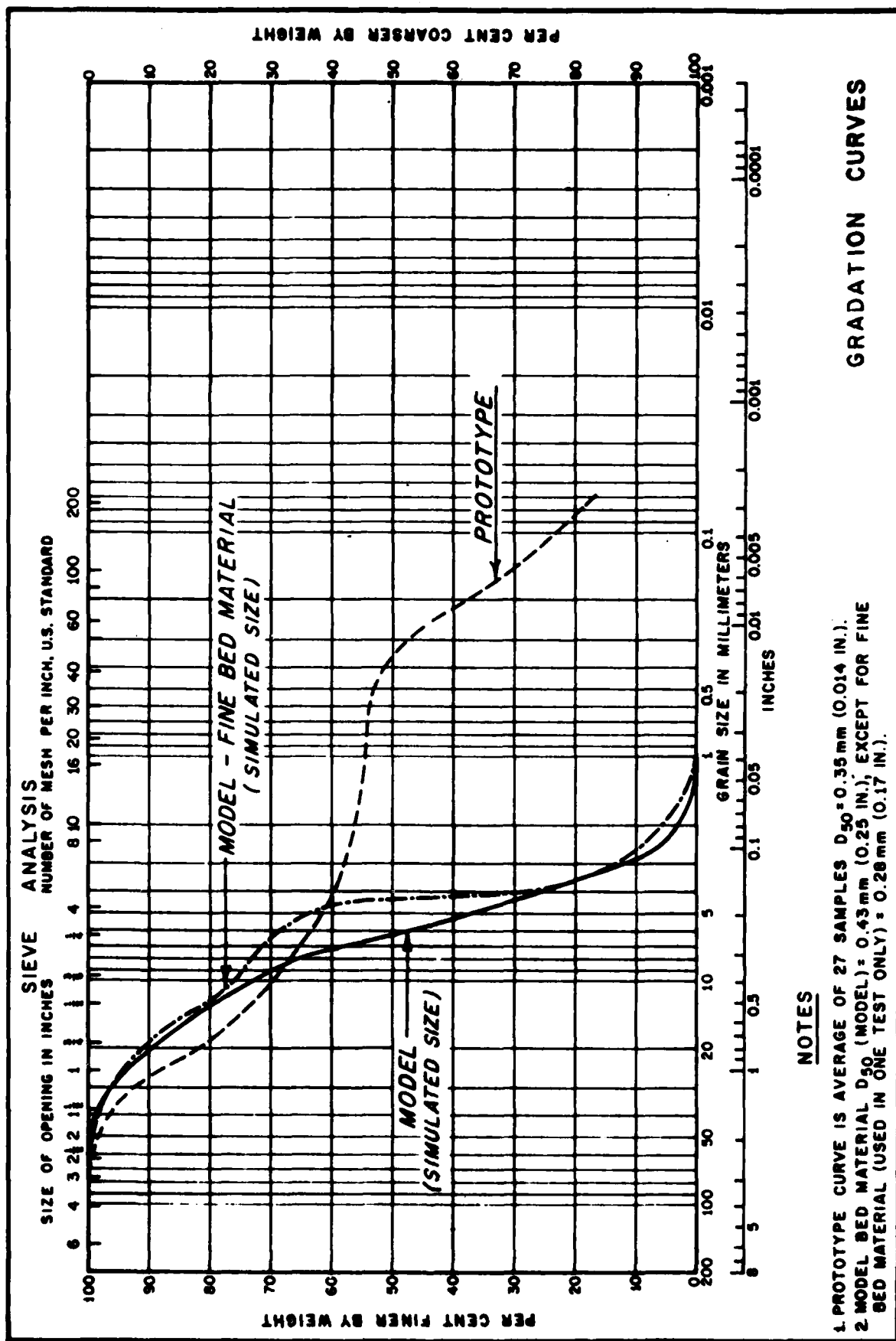
1. DETAILS OF OVERFLOW SILL AND BASIN SHOWN ON PLATE 49.
2. BED MATERIAL, SAND $d_{50} = 0.43\text{mm}$ (MODEL).

DIVERSION CHANNEL SCOUR AT OVERFLOW SILL PLAN B

FLOODWAY DISCHARGES 74 000 AND 160 000 CFS

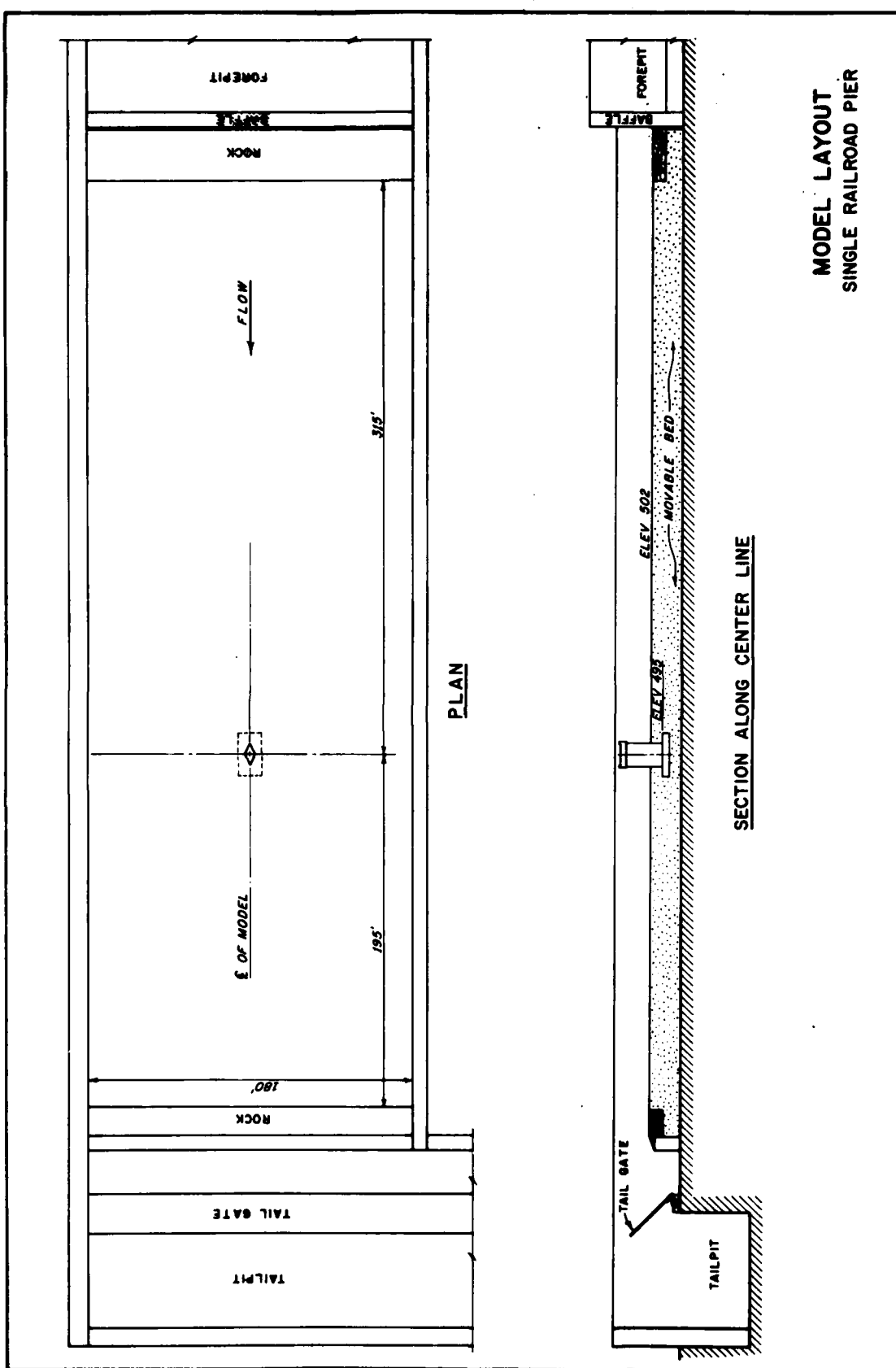
SCALE

0 100 200 300 400 500 FT

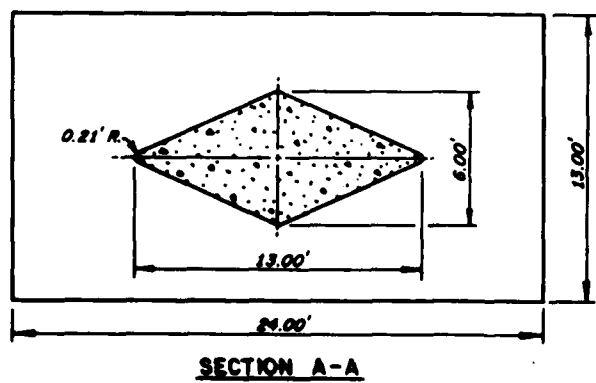
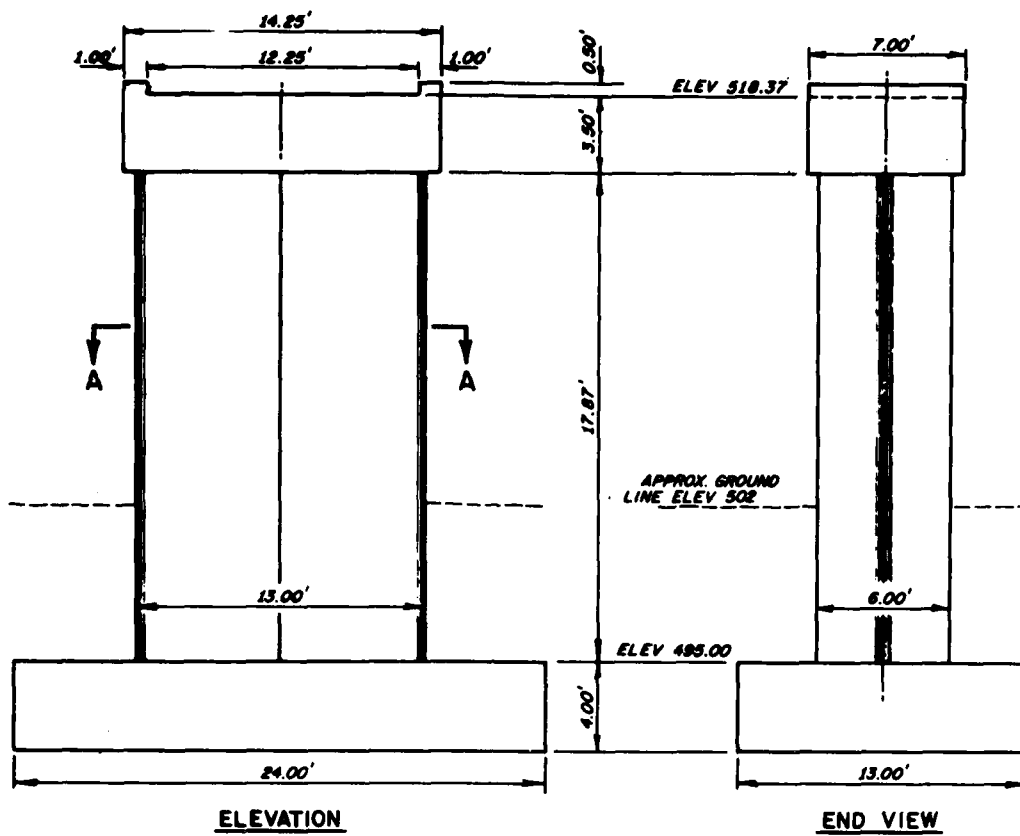


NOTES

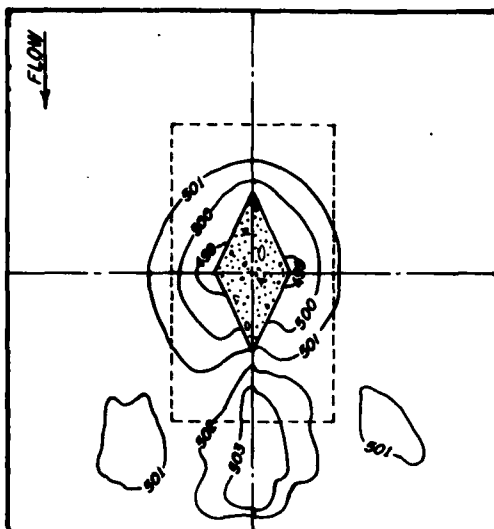
1. PROTOTYPE CURVE IS AVERAGE OF 27 SAMPLES $D_{50} = 0.35 \text{ mm (0.014 IN.)}$.
2. MODEL BED MATERIAL D_{50} (MODEL) = $0.43 \text{ mm (0.25 IN.)}$, EXCEPT FOR FINE BED MATERIAL (USED IN ONE TEST ONLY) = $0.28 \text{ mm (0.17 IN.)}$.



MODEL LAYOUT
SINGLE RAILROAD PIER

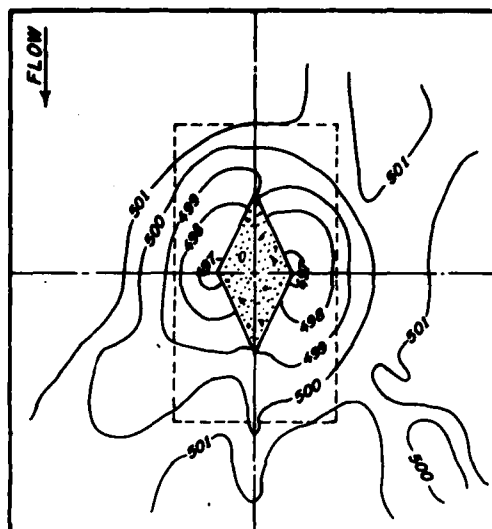


DETAILS OF RAILROAD PIER



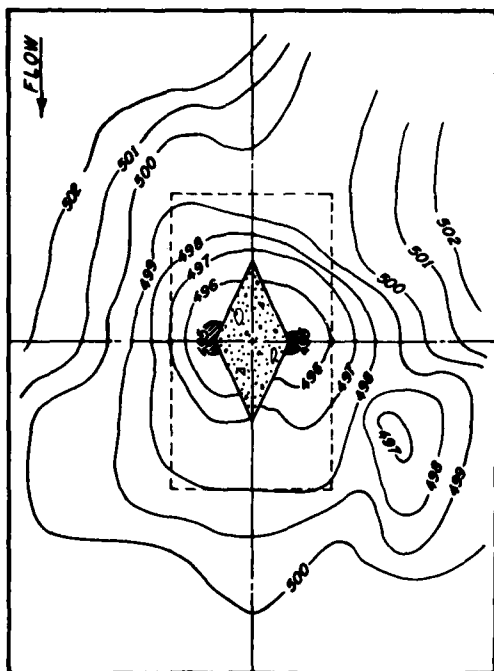
100-YEAR FLOOD

DISCHARGE: 40 CFS / FT
 APPROACH VELOCITY: 4.32 FPS
 DEPTH OF FLOW 9.28 FT
 DURATION OF TEST: 3 HRS 52 MIN



STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.70 FPS
 DEPTH OF FLOW 11.92 FT
 DURATION OF TEST: 1 HR 56 MIN.



PROBABLE MAXIMUM FLOOD

DISCHARGE: 130 CFS / FT
 APPROACH VELOCITY: 9.30 FPS
 DEPTH OF FLOW 14.91 FT
 DURATION OF TEST: 1 HR 56 MIN

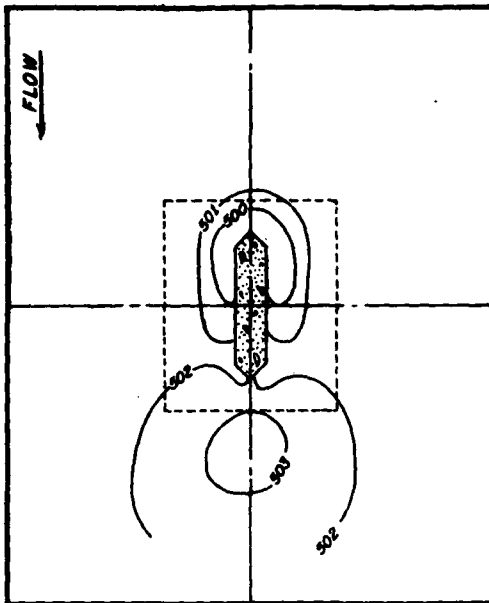
NOTES

1. MODEL LAYOUT AND PIER DETAILS ARE SHOWN ON PLATES 77 AND 78.
2. BED AT START OF TEST, ELEV 502.
3. AVERAGE DIAMETER OF BED MATERIAL, 0.25 IN.
4. PIER FOOTING UNPROTECTED.

SCALE

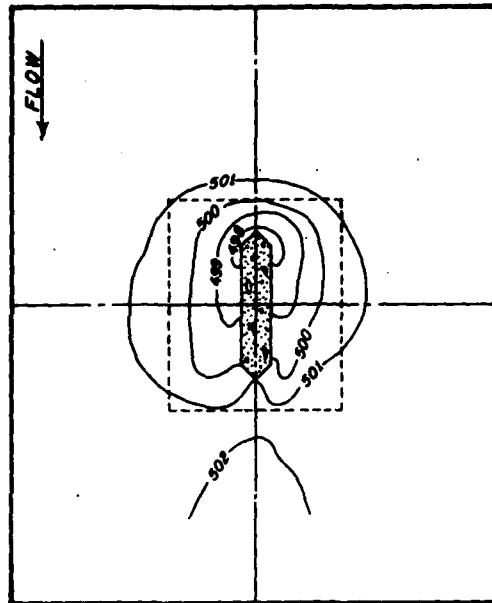


LOCAL SCOUR PATTERNS RAILROAD PIER



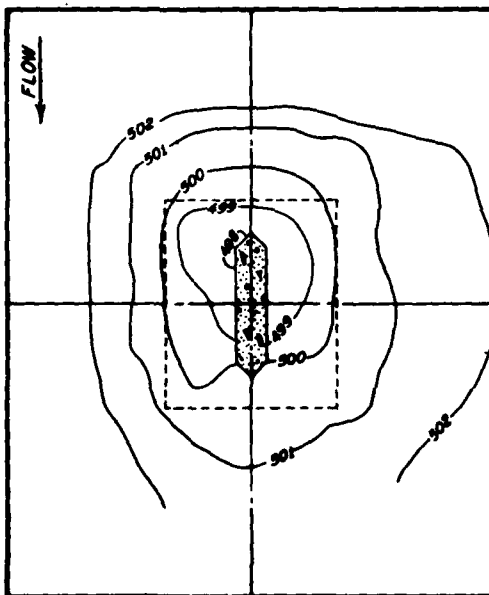
100-YEAR FLOOD

DISCHARGE: 44 CFS / FT
 APPROACH VELOCITY: 4.50 FPS
 DEPTH OF FLOW 9.84 FT
 DURATION OF TEST: 1 HR 56 MIN



STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 36 MIN.



PROBABLE MAXIMUM FLOOD

DISCHARGE: 147 CFS / FT
 APPROACH VELOCITY: 8.60 FPS
 DEPTH OF FLOW 17.10 FT
 DURATION OF TEST: 1 HR 56 MIN

NOTES

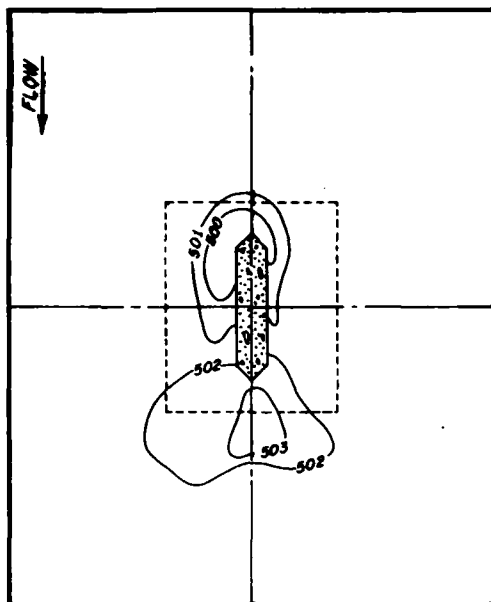
1. MODEL LAYOUT AND PIER DETAILS ARE SHOWN ON PLATES 80 AND 81.
2. BED AT START OF TEST, ELEV 502.
3. AVERAGE DIAMETER OF BED MATERIAL, 0.25 IN.
4. PIER FOOTING UNPROTECTED.

SCALE



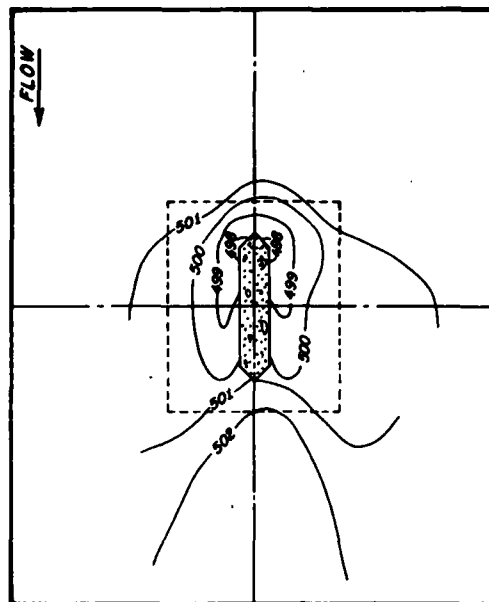
LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 UPSTREAM PIER



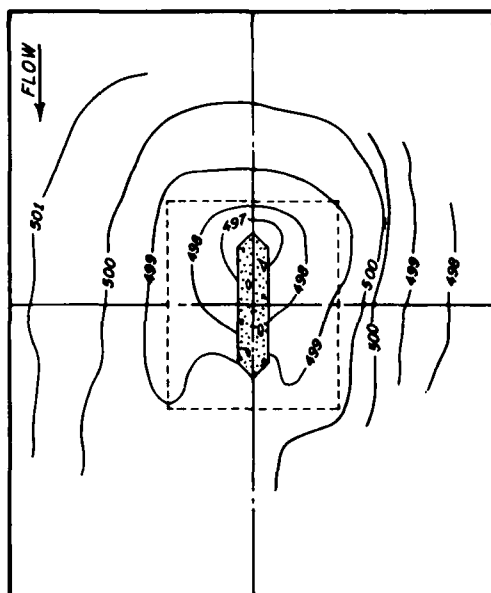
100-YEAR FLOOD

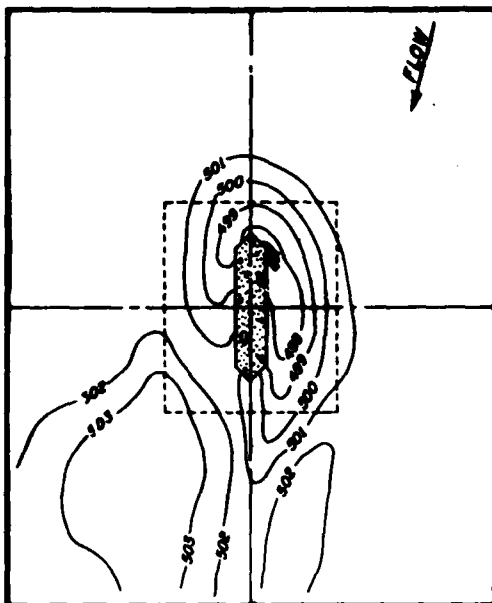
DISCHARGE: 44 CFS / FT
 APPROACH VELOCITY: 4.50 FPS
 DEPTH OF FLOW 9.84 FT
 DURATION OF TEST: 1 HR 56 MIN



STANDARD PROJECT FLOOD

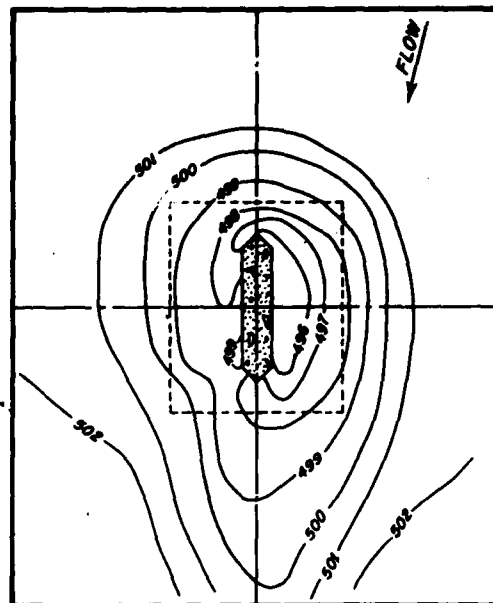
DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 36 MIN.





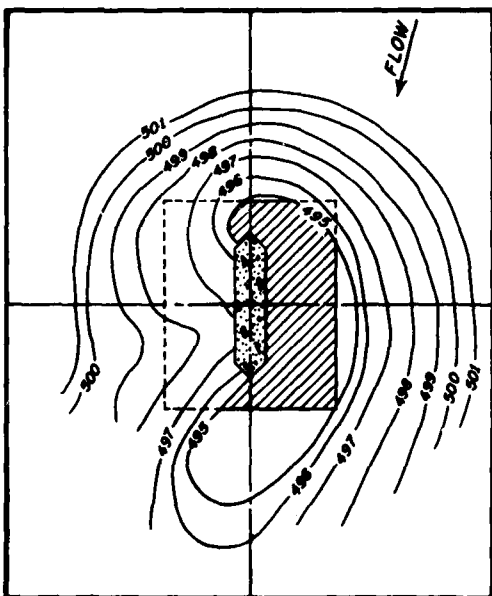
100-YEAR FLOOD

DISCHARGE: 44 CFS / FT
 APPROACH VELOCITY: 4.50 FPS
 DEPTH OF FLOW 9.84 FT
 DURATION OF TEST: 1 HR 56 MIN



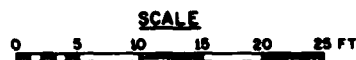
STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN.



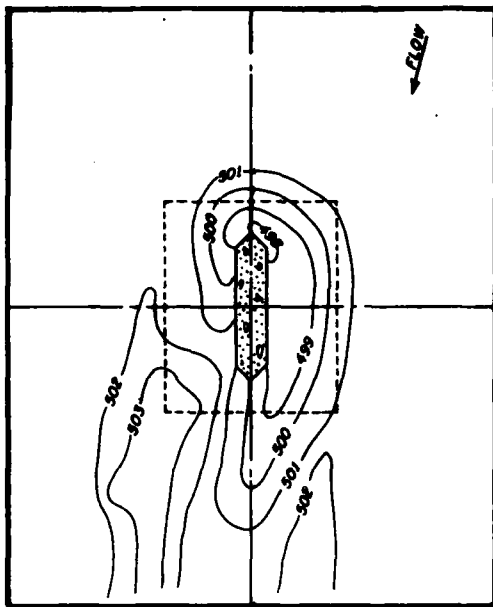
PROBABLE MAXIMUM FLOOD

DISCHARGE: 147 CFS / FT
 APPROACH VELOCITY: 8.80 FPS
 DEPTH OF FLOW 17.10 FT
 DURATION OF TEST: 2 HRS 15 MIN



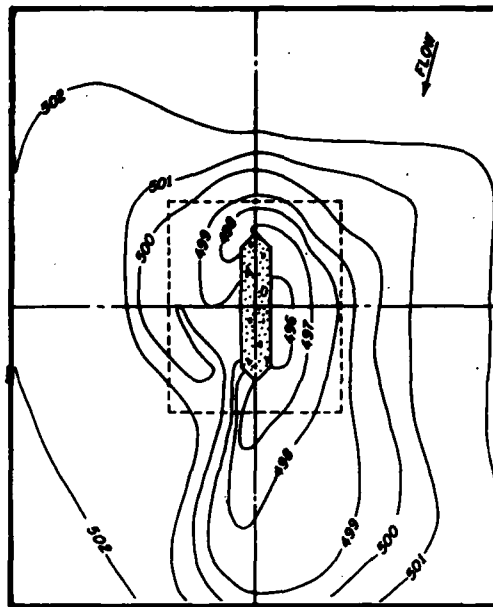
LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 UPSTREAM PIER
 15-DEGREE APPROACH FLOW



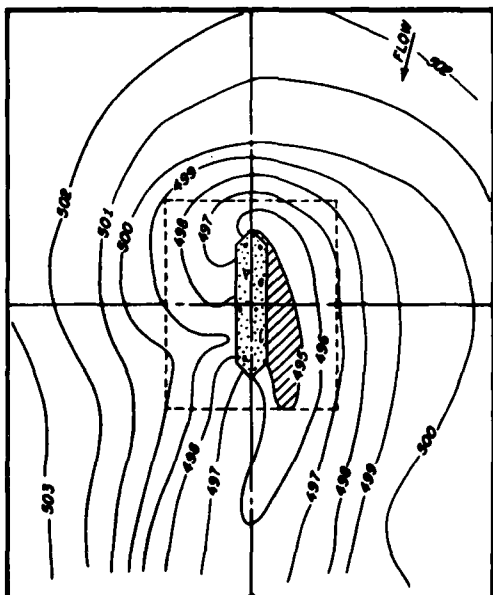
100-YEAR FLOOD

DISCHARGE: 44 CFS / FT
 APPROACH VELOCITY: 4.50 FPS
 DEPTH OF FLOW 9.84 FT
 DURATION OF TEST: 1 HR 56 MIN



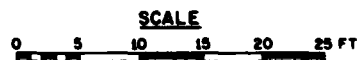
STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN



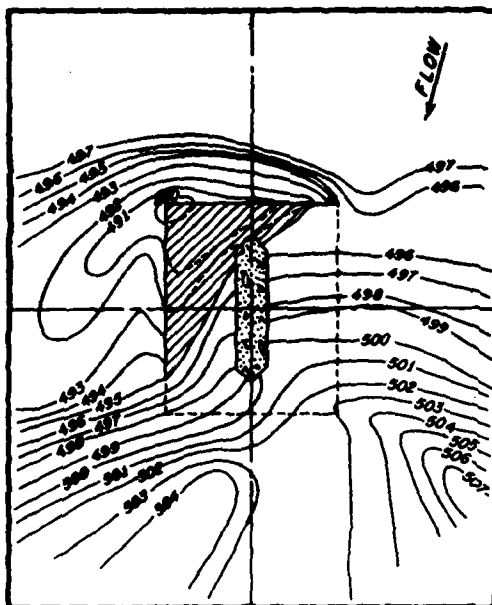
PROBABLE MAXIMUM FLOOD

DISCHARGE: 147 CFS / FT
 APPROACH VELOCITY: 8.60 FPS
 DEPTH OF FLOW 17.30 FT
 DURATION OF TEST: 2 HRS 15 MIN

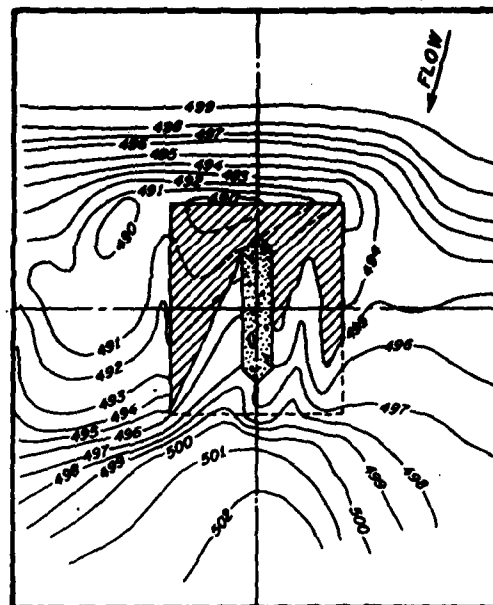


LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 DOWNSTREAM PIER
 15-DEGREE APPROACH FLOW



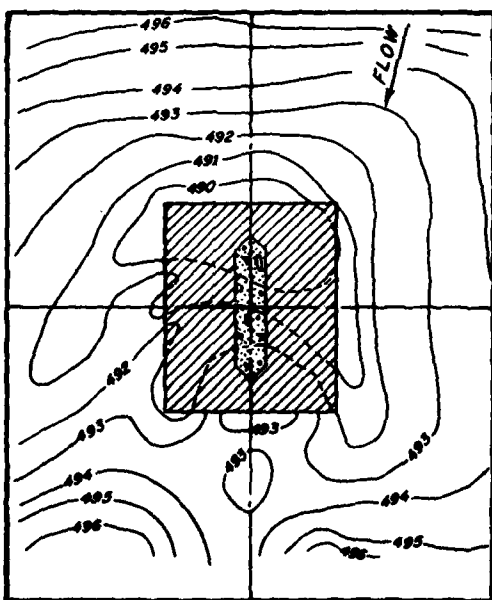
RUN 1



RUN 2

STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN



PROBABLE MAXIMUM FLOOD

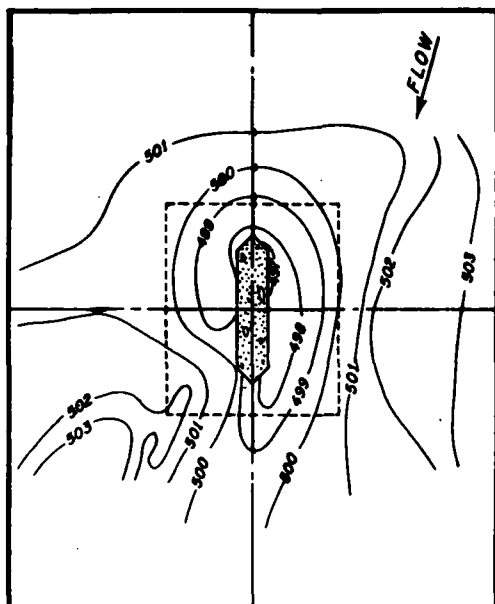
DISCHARGE: 147 CFS / FT
 APPROACH VELOCITY: 8.60 FPS
 DEPTH OF FLOW 17.10 FT
 DURATION OF TEST: 1 HR 56 MIN



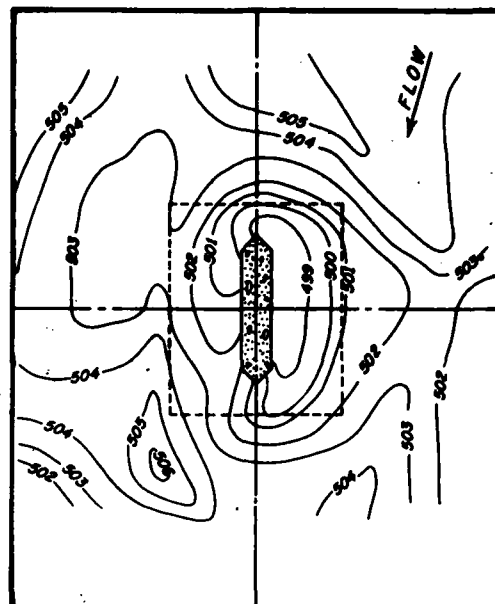
DEBRIS ON UPSTREAM PIERS

LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 UPSTREAM PIER
 15-DEGREE APPROACH FLOW



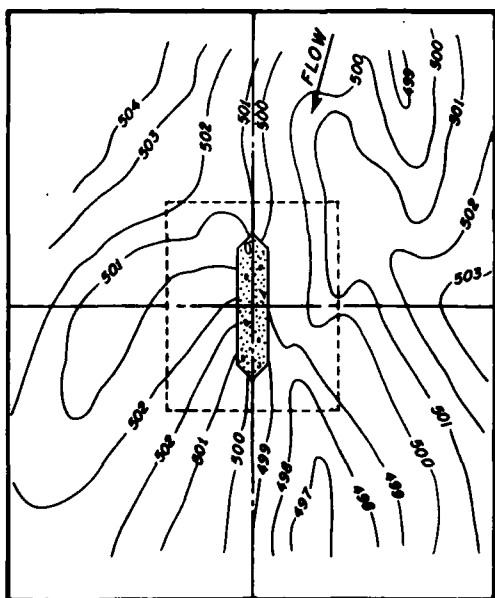
RUN 1



RUN 2

STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW: 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN



PROBABLE MAXIMUM FLOOD

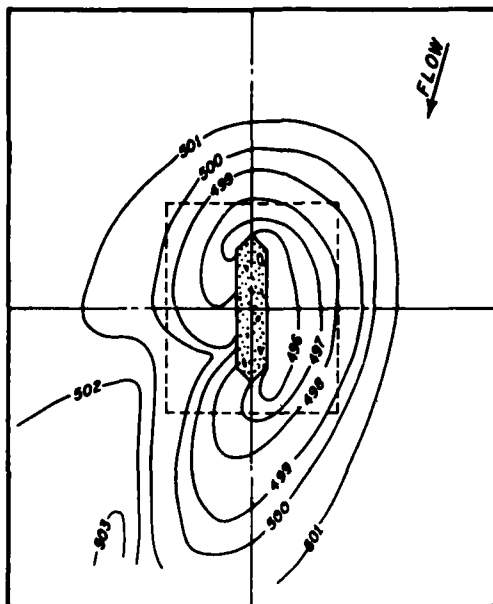
DISCHARGE: 147 CFS / FT
 APPROACH VELOCITY: 8.60 FPS
 DEPTH OF FLOW: 17.10 FT
 DURATION OF TEST: 1 HR 56 MIN



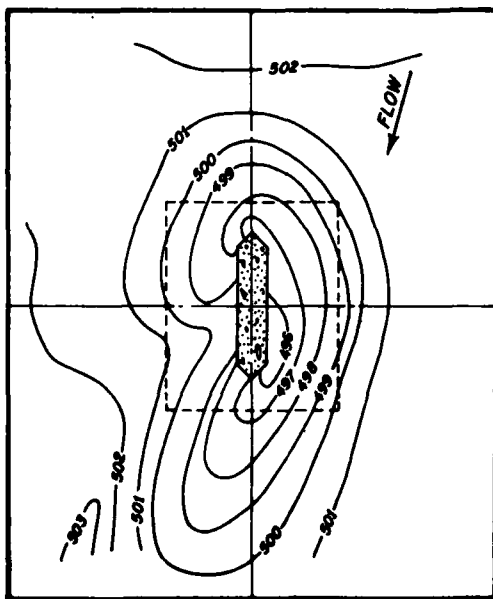
DEBRIS ON UPSTREAM PIERS

LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 DOWNSTREAM PIER
 15-DEGREE APPROACH FLOW



UPSTREAM PIER



DOWNSTREAM PIER



FINE BED MATERIAL

STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN

LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 15-DEGREE APPROACH FLOW

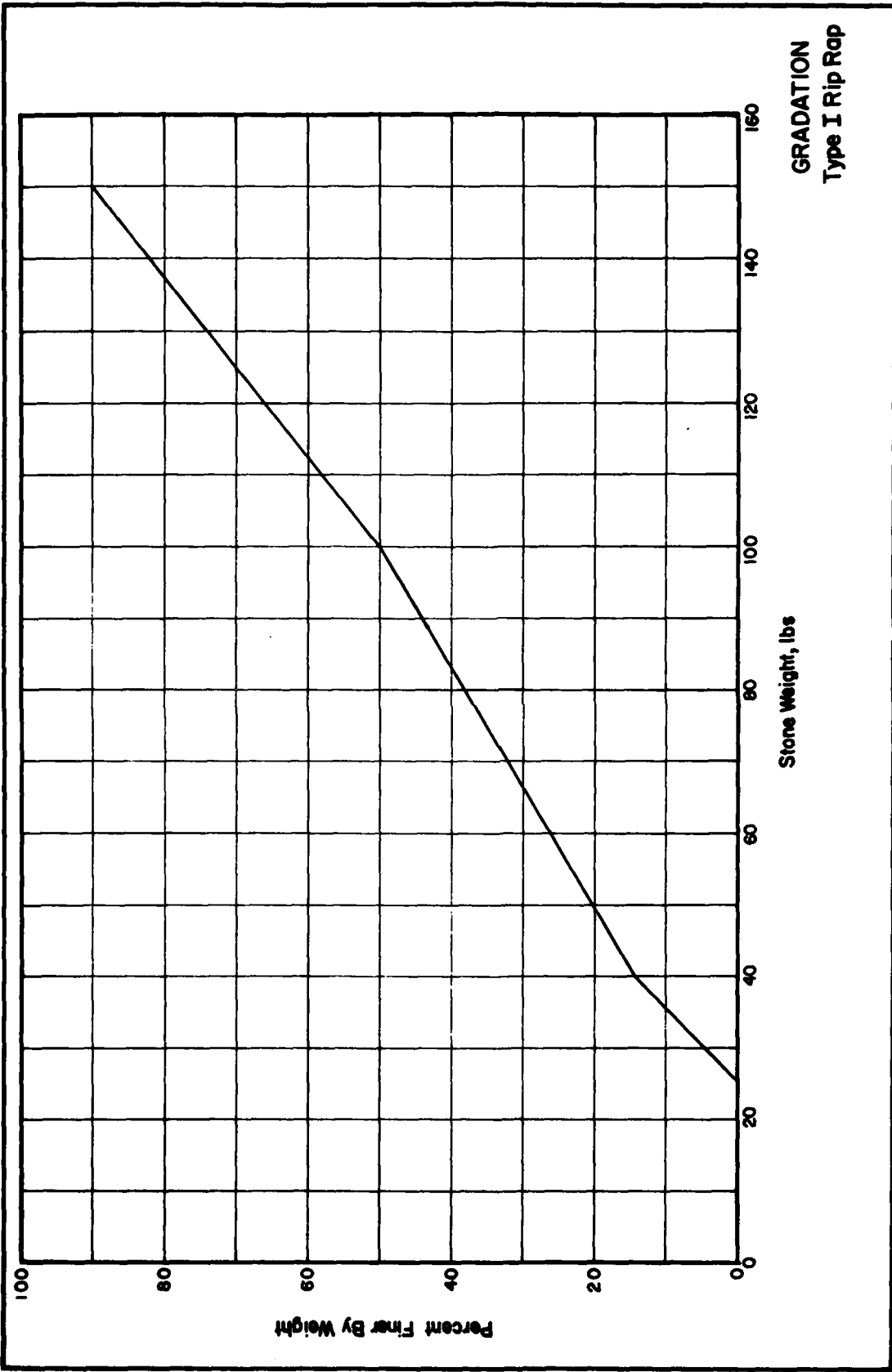
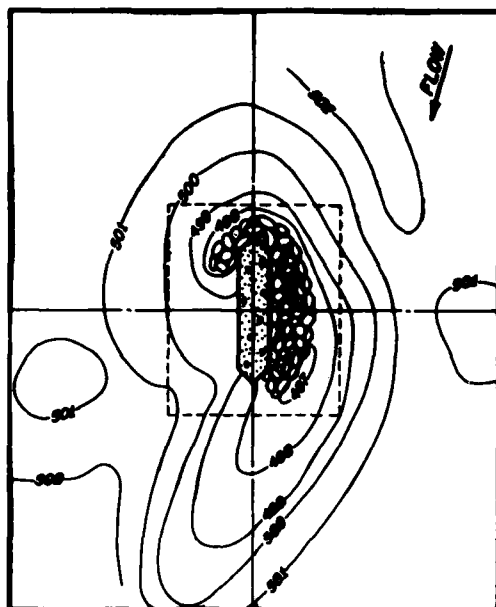
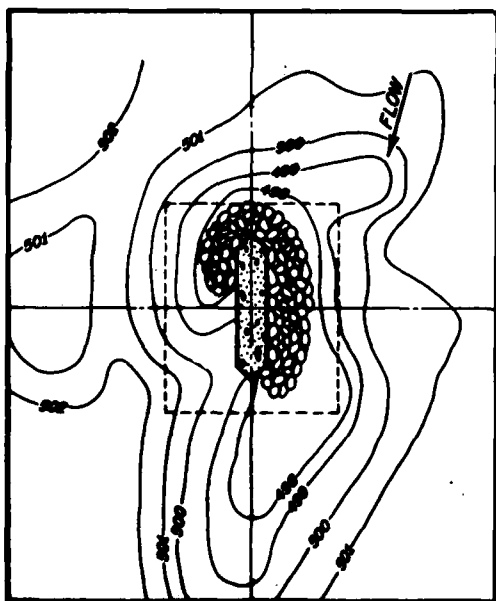


PLATE 89

2



UPSTREAM PIER



DOWNSTREAM PIER



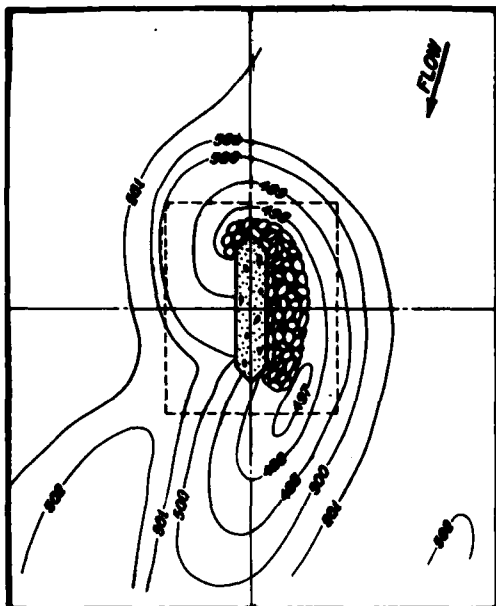
TYPE I ROCK PROTECTION

STANDARD PROJECT FLOOD

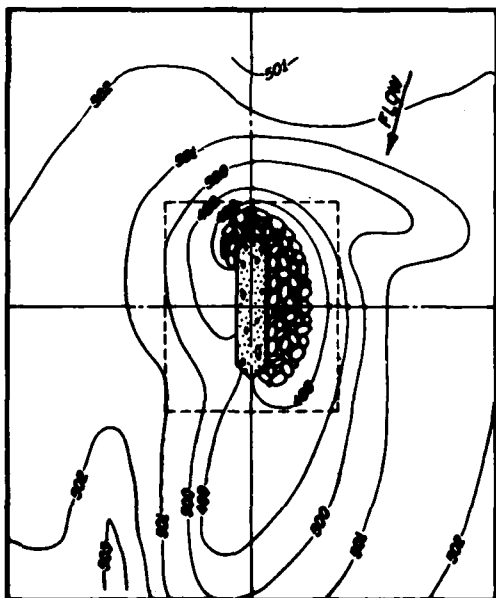
DISCHARGE: 80 CFS / FT
 APPROACH VELOCITY: 6.39 FPS
 DEPTH OF FLOW 12.60 FT
 DURATION OF TEST: 1 HR 56 MIN

LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
 15-DEGREE APPROACH FLOW



UPSTREAM PIER



DOWNSTREAM PIER

SCALE

0 5 10 15 20 25 FT

CLEAR-WATER SCOUR
TYPE I ROCK PROTECTION

STANDARD PROJECT FLOOD

DISCHARGE: 80 CFS / FT
APPROACH VELOCITY: 6.39 FPS
DEPTH OF FLOW: 12.80 FT
DURATION OF TEST: 1 HR 55 MIN

LOCAL SCOUR PATTERNS

SIX HIGHWAY PIERS
15-DEGREE APPROACH FLOW

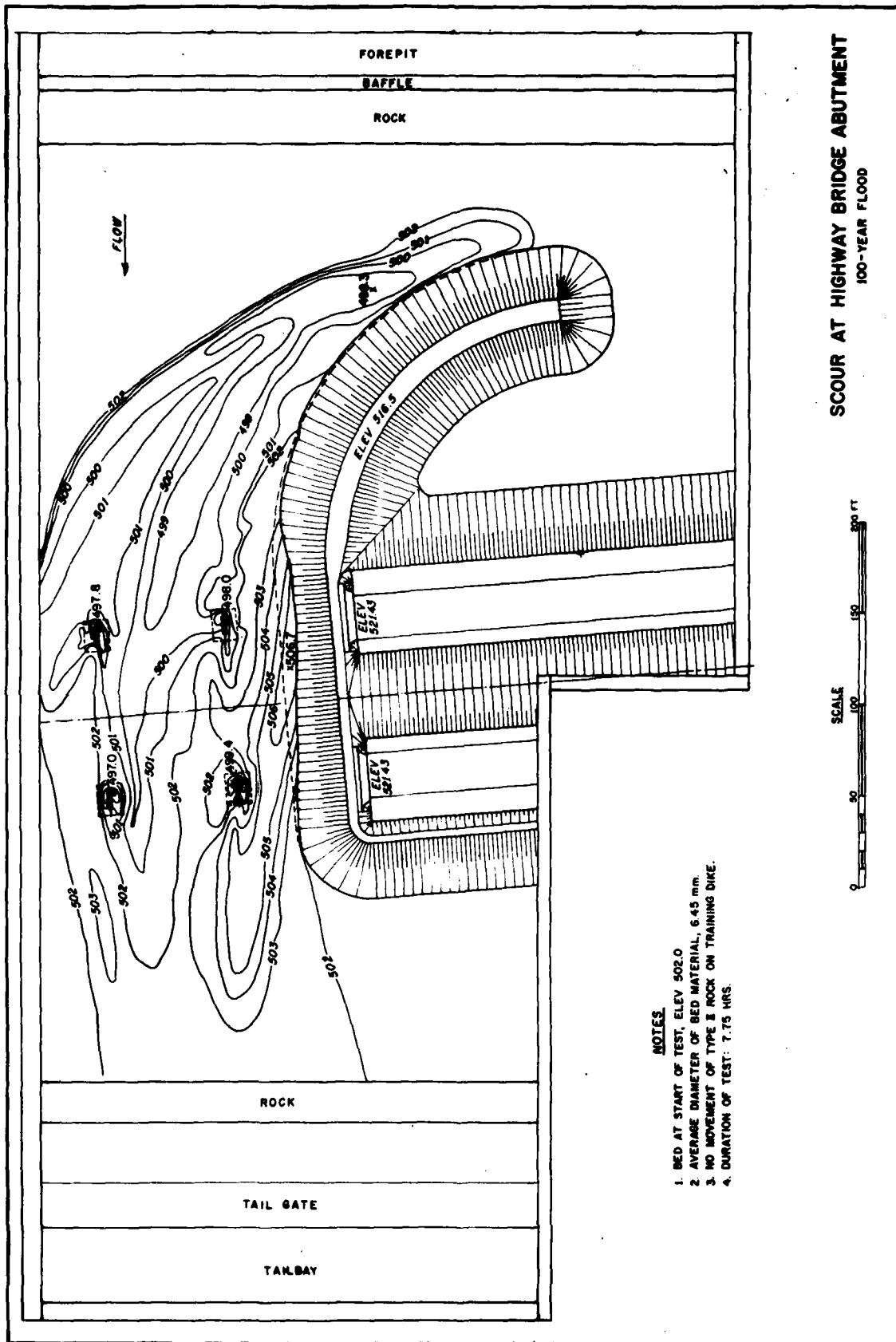


PLATE 93

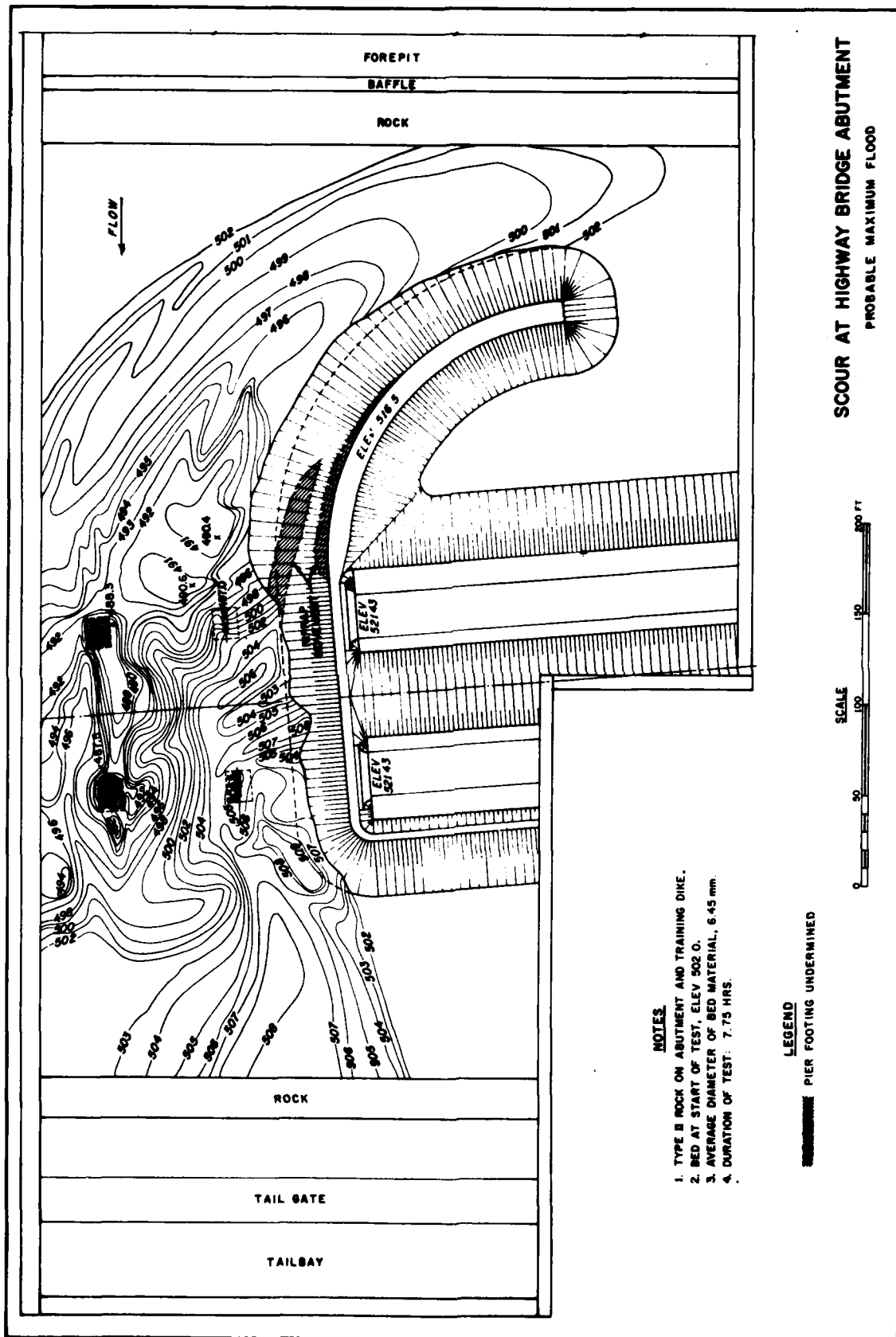
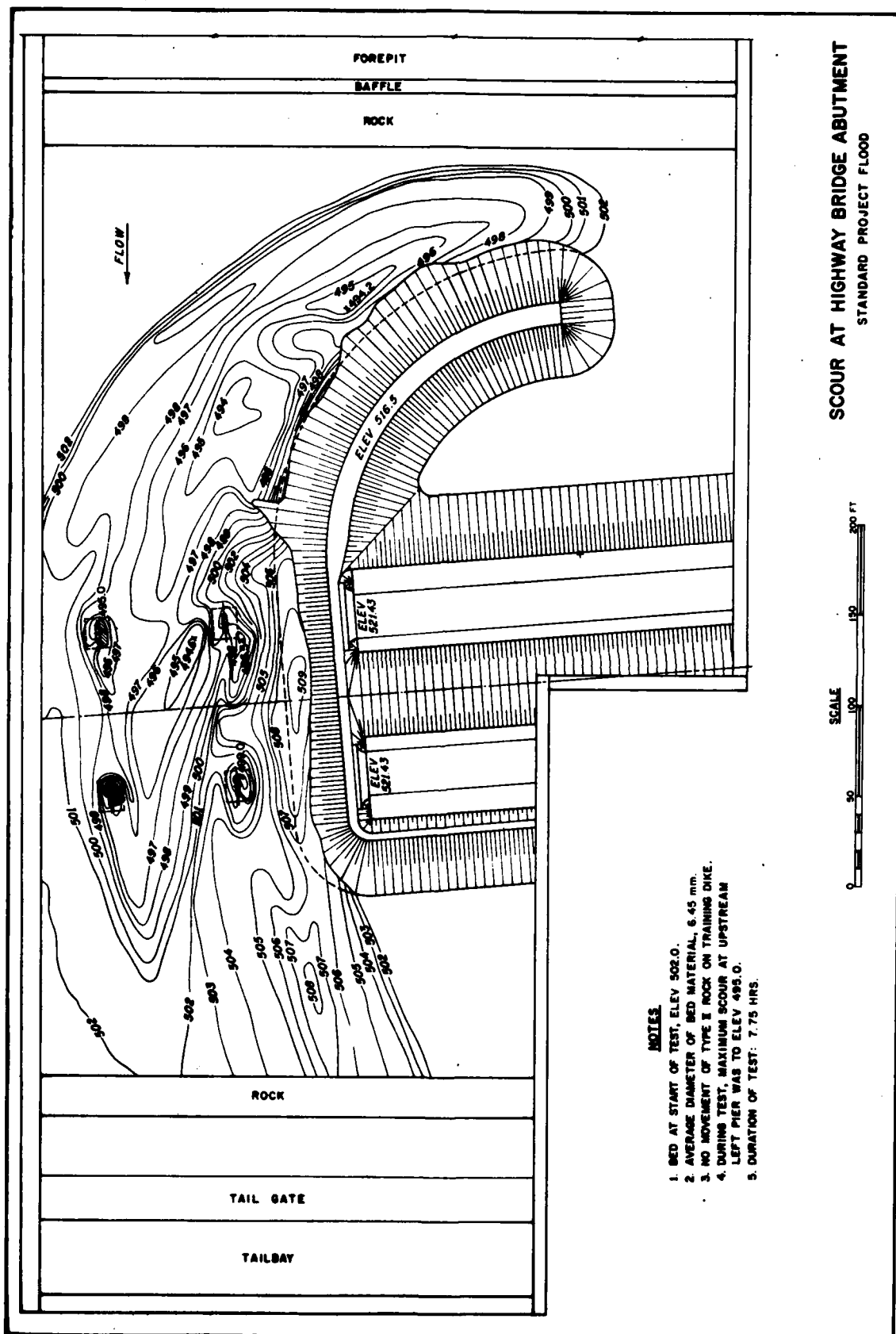
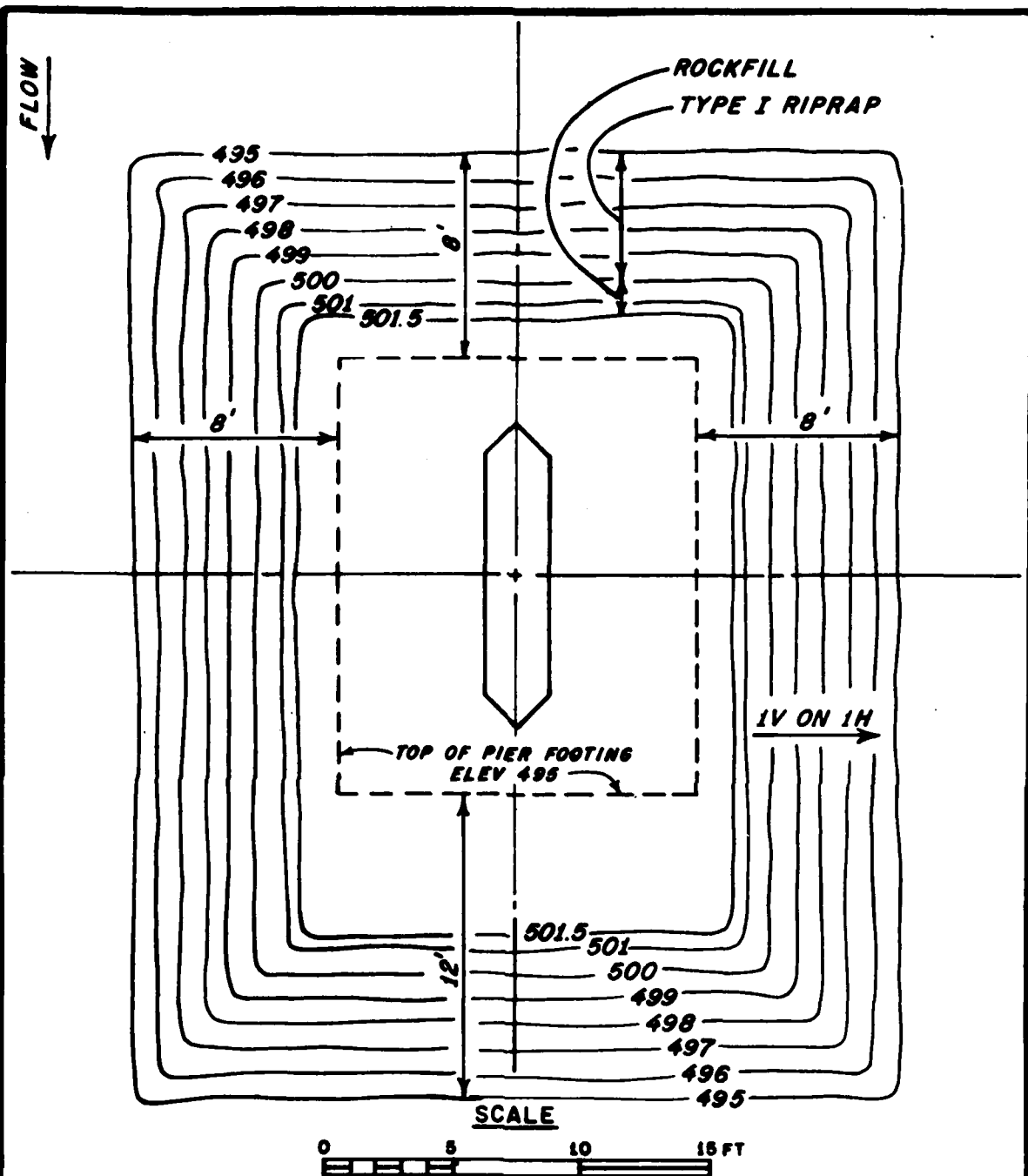


PLATE 95





NOTES

1. PIER DETAILS SHOWN ON PLATE 81.
2. ROCK PROTECTION BACKFILLED TO ELEV 502.

ROCK PROTECTION
HIGHWAY PIER

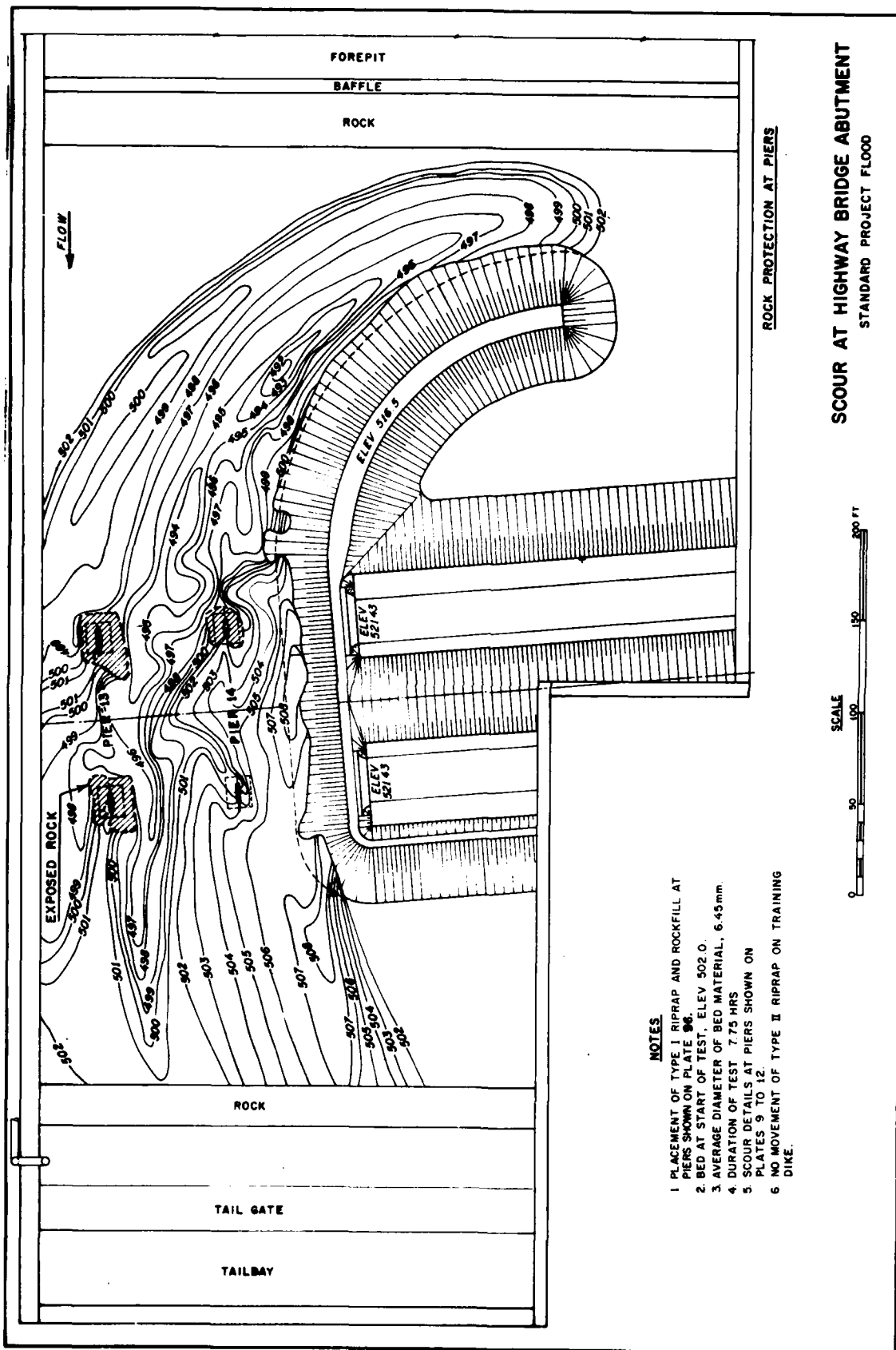
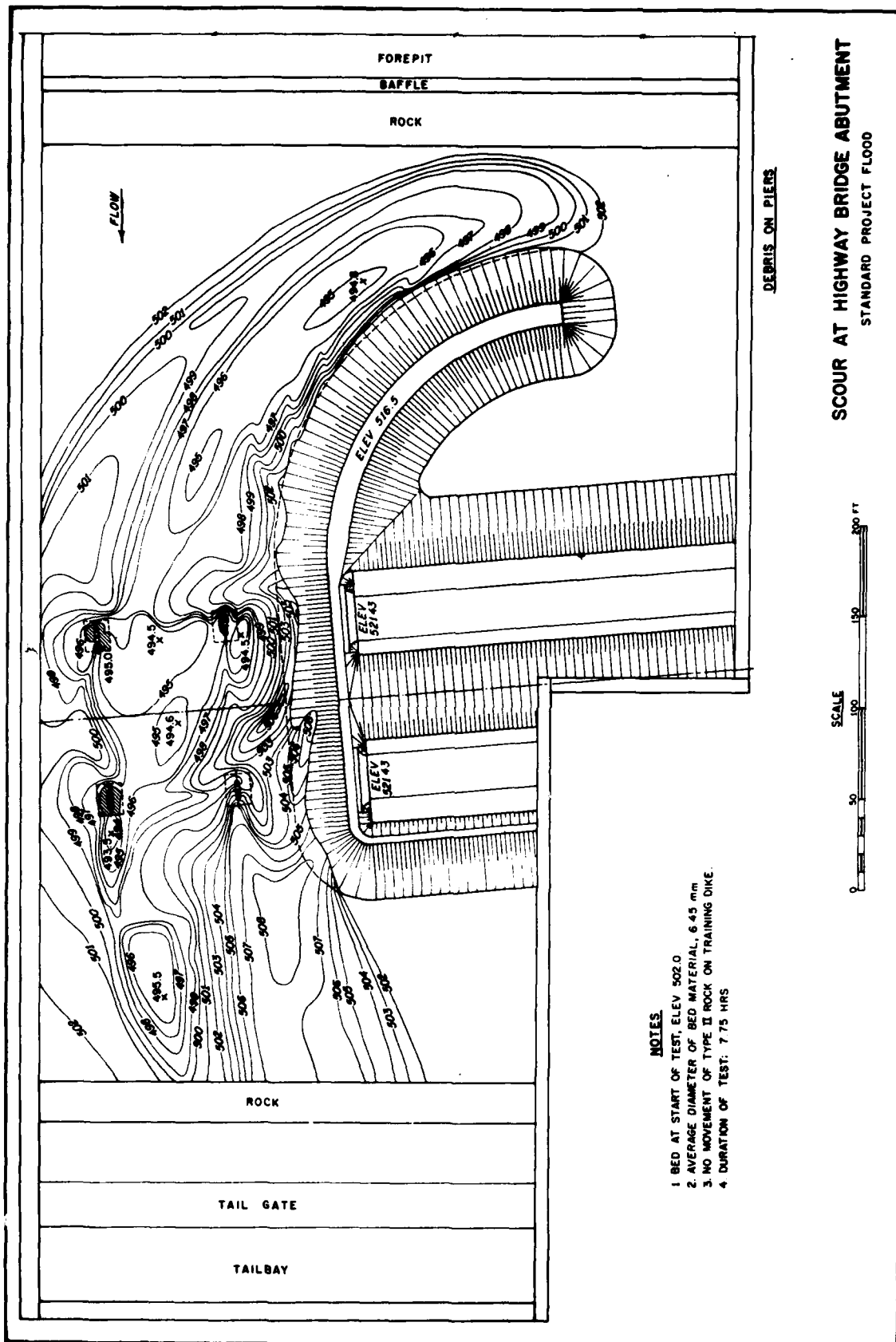
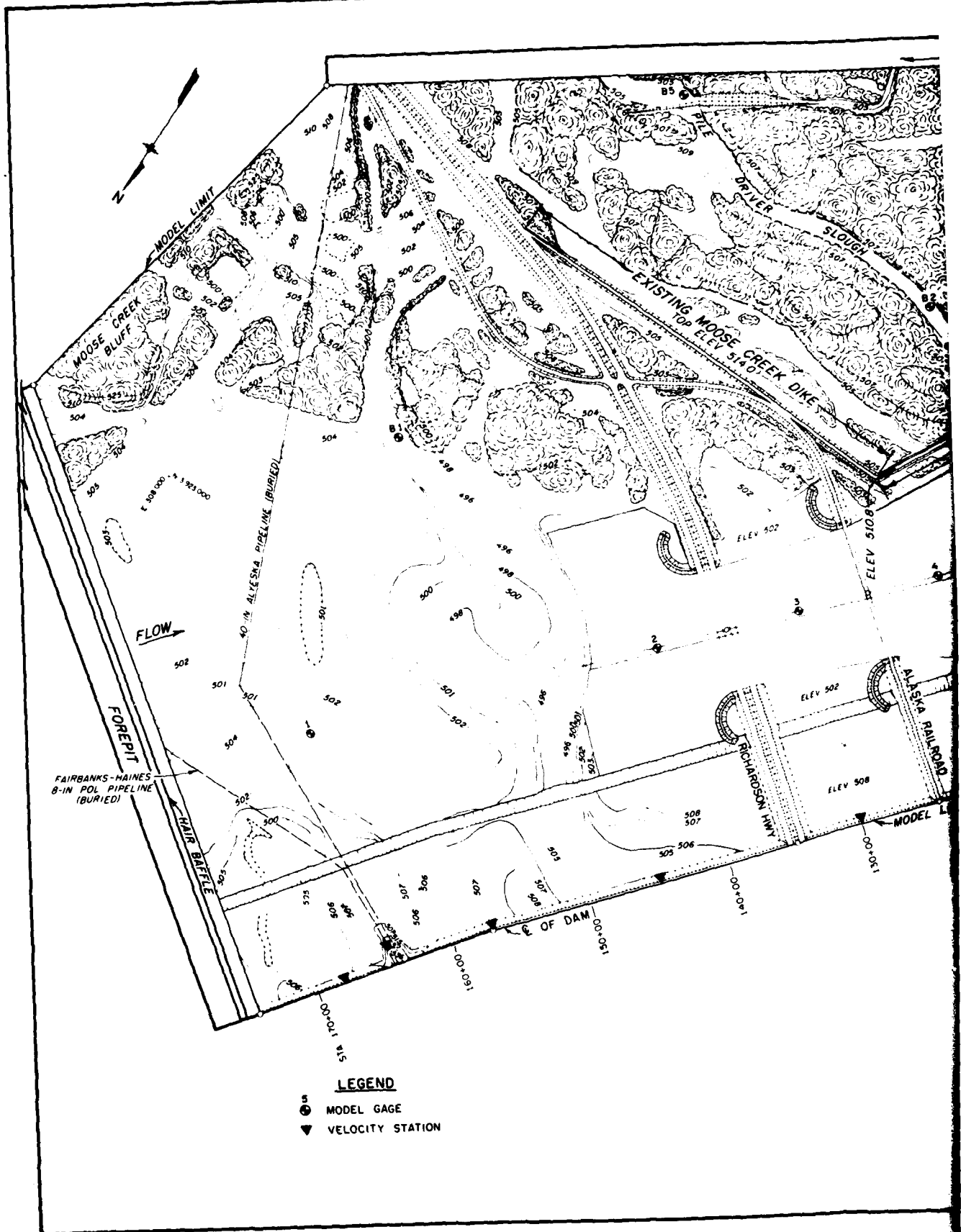
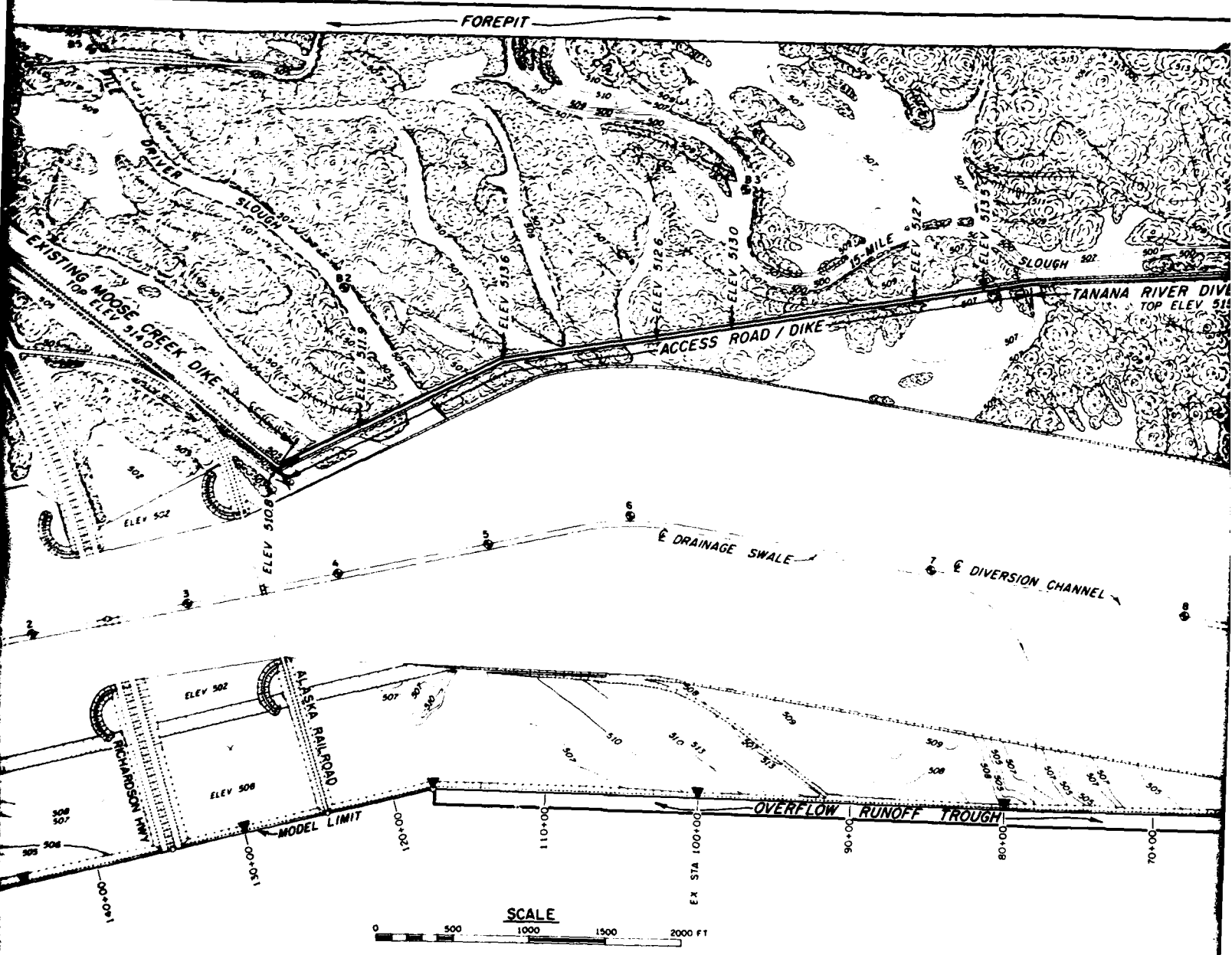


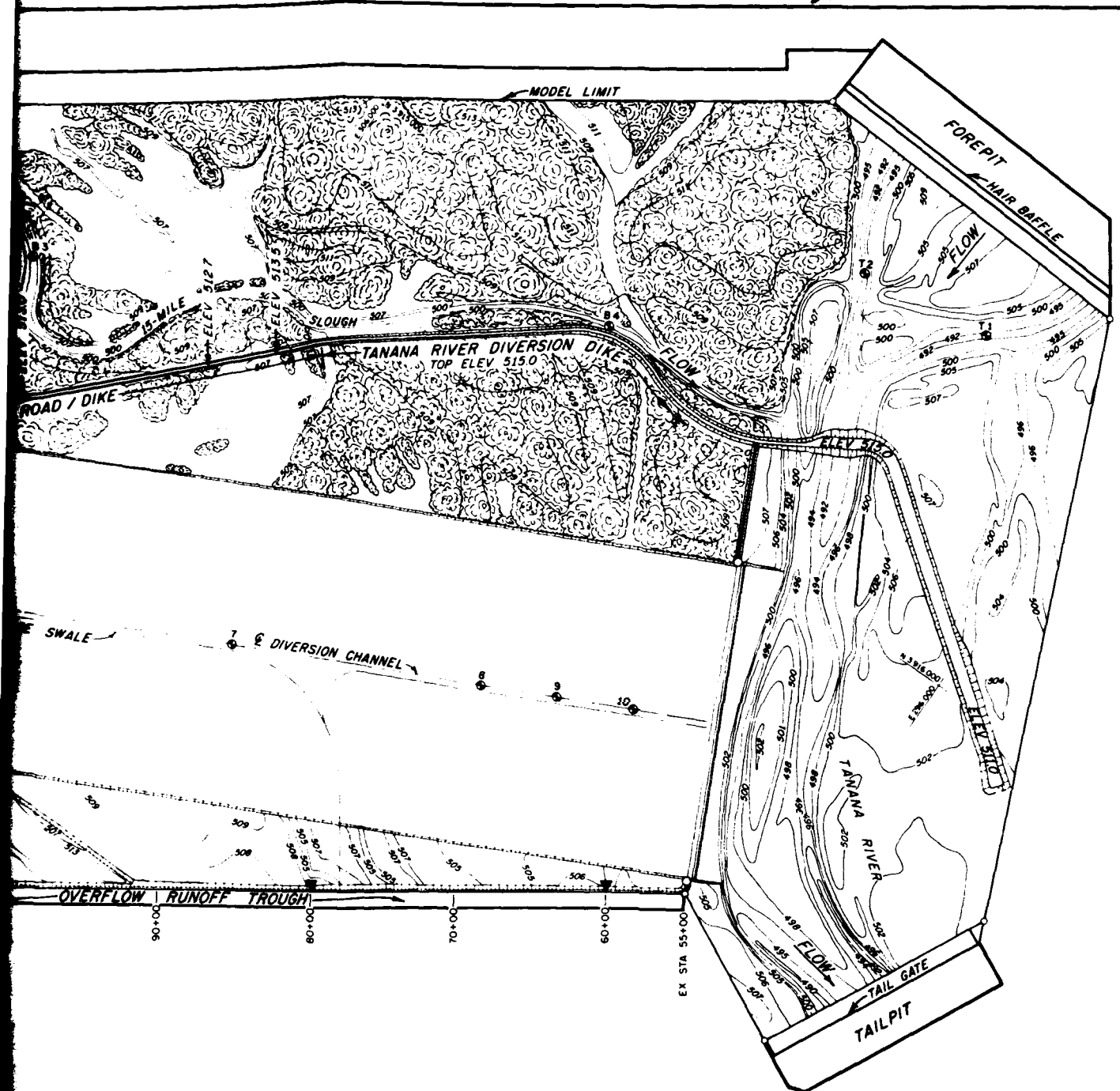
PLATE 97







2



MODIFIED DIVERSION CHANNEL
PLAN B WITH ACCESS ROAD/DIKE

1

3