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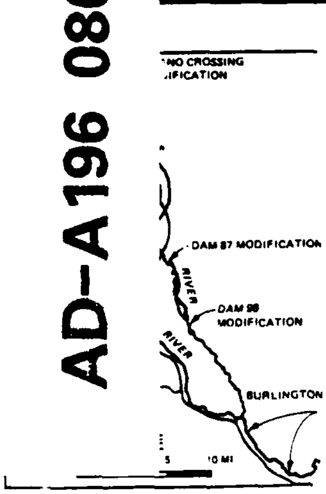


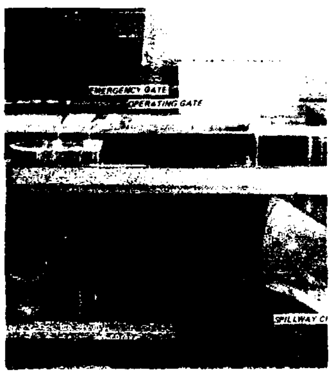

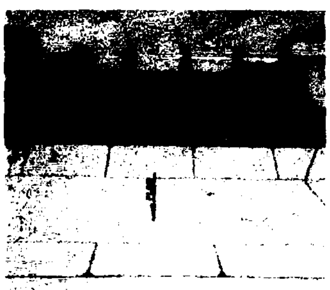

TECHNICAL REPORT HL-88-9

# LAKE DARLING SPILLWAY SOURIS RIVER, NORTH DAKOTA

## Hydraulic Model Investigation

by

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The concrete gravity ogee spillway for the Lake Darling Dam will contain five 22-ft-high by 43-ft-wide gate bays separated by 10-ft-wide piers. Outlet works consisting of four sluices within the piers will discharge into the spillway stilling basin. A hydraulic-jump-type stilling basin will provide satisfactory dissipation of the spillway and sluice flows. A 1:36-scale model of the spillway, sluices, stilling basin, approach area, and exit channel was used to study the overall hydraulic performance of the structure. Tests indicated that flow conditions in the approach to the spillway with both uncontrolled and controlled flows were satisfactory for the full range of anticipated discharges. Eddies developed on either side of the stilling basin. Constricting the channel flare to a uniform width eliminated the eddies. The size and extent of stone protection required on the dam embankment, side slopes and bottom of the approach channel, and side slopes and bottom of the exit channel were determined by model investigations.

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# PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers (OCE), US Army, on 7 October 1983, at the request of the US Army Engineer District, St. Paul (NCS).

The studies were conducted by personnel of the Hydraulics Laboratory, US Army Engineer Waterways Experiment Station (WES), during the period October 1983 to September 1984. All studies were conducted under the direction of Messrs. H. B. Simmons and F. A. Herrmann, Jr., former and present Chiefs, Hydraulics Laboratory, and J. L. Grace, Jr., Chief, Hydraulic Structures Division. Tests were conducted by Mrs. D. R. Cooper and Messrs. E. L. Jefferson and R. Bryant, Jr., Spillways and Channels Branch, under the supervision of Mr. N. R. Oswalt, Chief, Spillways and Channels Branch. This report was prepared by Mrs. Cooper and edited by Mrs. Nancy Johnson, Information Technology Laboratory, under the Inter-Governmental Personnel Act.

During the course of the investigation, Messrs. S. Powell and T. Munsey, OCE; J. Ordonez, US Army Engineer Division, North Central; and J. Murphy, M. Ziemer, and D. Reinartz, NCS, visited WES to discuss test results and correlate these results with current design studies.

A special tribute is made to J. L. Grace, Jr., for his excellent technical guidance on this project.

COL Dwayne G. Lee, CE, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

## CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| cubic feet | 0.02831685 | cubic metres |
| degrees (angular) | 0.01745329 | radians |
| feet | 0.3048 | metres |
| feet of water (39.2° F) | 2.98898 | kilopascals |
| inches | 25.4 | millimetres |
| miles (US statute) | 1.609344 | kilometres |
| pounds (mass) | 0.4535924 | kilograms |
| pounds (mass) per cubic foot | 16.01846 | kilograms per cubic metre |

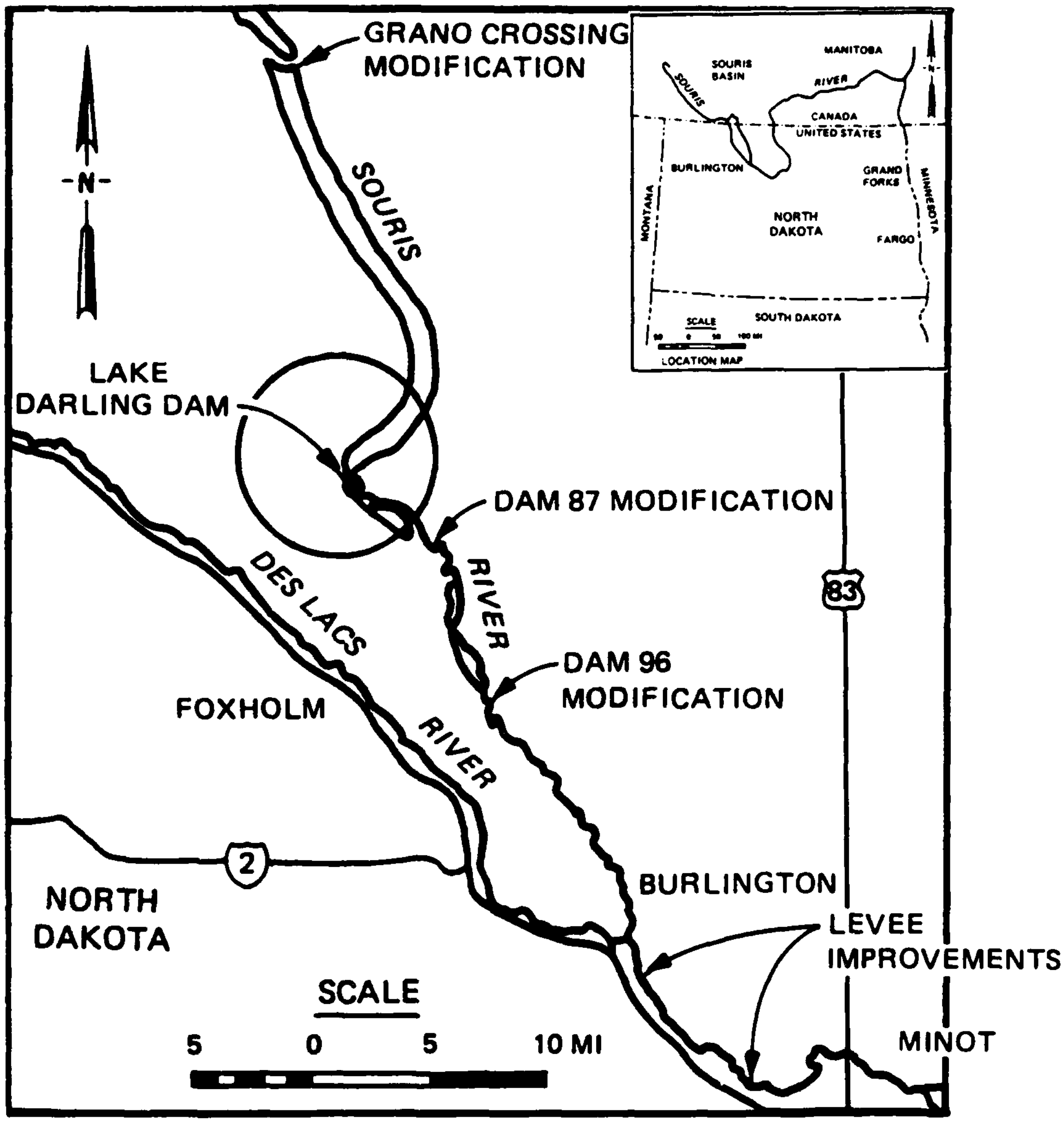


Figure 1. Location map

LAKE DARLING SPILLWAY

SOURIS RIVER, NORTH DAKOTA

Hydraulic Model Investigation

# PART I: INTRODUCTION

## The Prototype

1. Lake Darling (Figure 1) is a large storage reservoir created by a dam northwest of Minot, North Dakota, at the Ward-Renville County line on the Souris River (mile 429.9). The reservoir extends 27 miles* up a valley. The project is one unit in the Upper Souris National Wildlife Refuge and after modification will impound water for flood control and favorable waterfowl conditions downstream. Lake Darling is classified as a large dam; therefore, the spillway design flood is the probable maximum flood (PMF), which is 99,800 cfs.

2. The modified dam will consist of a concrete gravity-type spillway structure flanked by compacted earth-fill embankments to high ground on the east and west sides of the river. The total length of the concrete dam and earth embankments will be 3,170 ft. At the top of the structure (el 1,614.0**), the length of the earth portion of the dam will be about 2,915 ft. The general plan and profile of the portion of the dam investigated in the model study with model limits are shown in Plate 1.

3. The outlet works will consist of four rectangular galvanized steel conduits 4 ft high, 3 ft wide, and 90 ft long with an intake invert elevation of 1,571.55 ft and an outlet invert elevation of 1,571.50 ft and will discharge into the spillway stilling basin (Plate 2). These low-flow conduits will be encased inside a 10-ft-wide pier. A trashrack will be provided at each sluice inlet. Each pier will contain two 4- by 4-ft rectangular wet wells and an air vent. The downstream wet well will contain a 3-ft-wide by 4-ft-high sluice gate to regulate flow.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

4. Flows exceeding storage capacity of the reservoir and discharge capacity of the outlet works will be passed over the gravity-type, concrete ogee spillway located in the river channel. The 255-ft-long spillway section with crest at el 1,584.0 will contain five tainter gates, each 43 ft wide and 22 ft high, between 10-ft-wide crest piers. Initial plans provided a hydraulic-jump-type basin, 80 ft long with two rows of 4.5-ft-high baffle piers, and a 2.2-ft-high sloped end sill (Plates 1 and 3).

## Purpose and Scope of the Model Study

5. Although the design of Lake Darling Spillway was based on sound hydraulic design practice, model analysis of its performance was desired to evaluate the long pier and sluice lengths and the basin action and to obtain discharge characteristics with combined operation of the sluices and the spillway tainter gates. The general spillway model investigation was particularly concerned with flow conditions in the approach and exit, spillway capacity, hydraulic performance of the stilling basin, height of training walls, and embankment and channel protective stone requirements. The sluice section model was concerned particularly with the hydraulic performance of the sluice operating singularly and in combination with the tainter gates.

## Presentation of Data

6. In the presentation of test results, no attempt is made to introduce the data in the chronological order in which the tests were conducted on the various models. Instead, as each element of the structure is considered, all tests conducted thereon are discussed in detail. All model data are presented in terms of prototype equivalents.

## PART II: THE MODEL

### Description

7. The comprehensive spillway model of Lake Darling Dam was constructed to a linear scale ratio of 1:36 and reproduced all topography and structures in an area extending 1,000 ft upstream, 1,700 ft downstream from the axis of the dam, 900 ft to the right, and 350 ft left of the center line of the spillway (Figure 2 and Plate 1). The portions of the model representing the approach, exit (including preformed scour hole), and overbank areas were molded of concrete to sheet metal templates and were given a brushed finish. The spillway, spillway gates, sluices and piers, and nonoverflow sections of the dam were constructed of sheet metal. The stilling basin, basin elements, and sidewalls were made of wood.

8. Supplementary tests of a sluice at a scale of 1:12 were made in the facility shown in Figure 3. The 1:12-scale section model reproduced one 3-ft-wide by 4-ft-high conduit encased in a 92-ft-long pier, an 8-ft-wide section of a gate bay on either side of the pier, and a 26-ft-wide section of the stilling basin. The tainter gates and spillway section were fabricated of noncorrugated metal. The stilling basin and baffle blocks were made of wood. The pier, sluice, and sluice gates were fabricated of plexiglas to permit better visual observation and evaluation of the hydraulic performance of the sluice.

9. Water used in the operation of the models was supplied by pumps, and discharges were measured with venturi meters. Steel rails set to grade along the sides of the flumes provided reference planes for measuring devices. Water-surface elevations were measured with point gages. Velocities were measured with a Montedoro Whitney model MVM-1 velocity meter (Figure 4). Current patterns were determined with dye injected into the water and confetti sprinkled on the water surface.

### Interpretation of Model Results

10. The accepted equations of hydraulic similitude, based upon the Froudian relations, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and the prototype. General

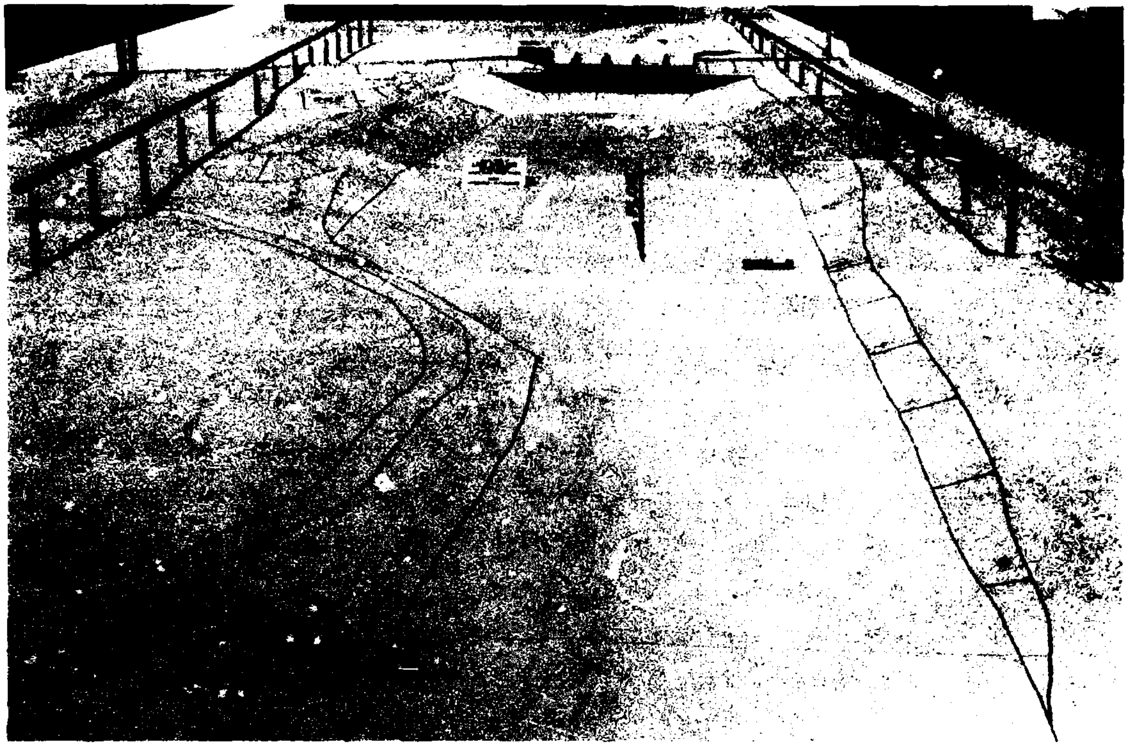

Figure 2. The 1:36-scale spillway model

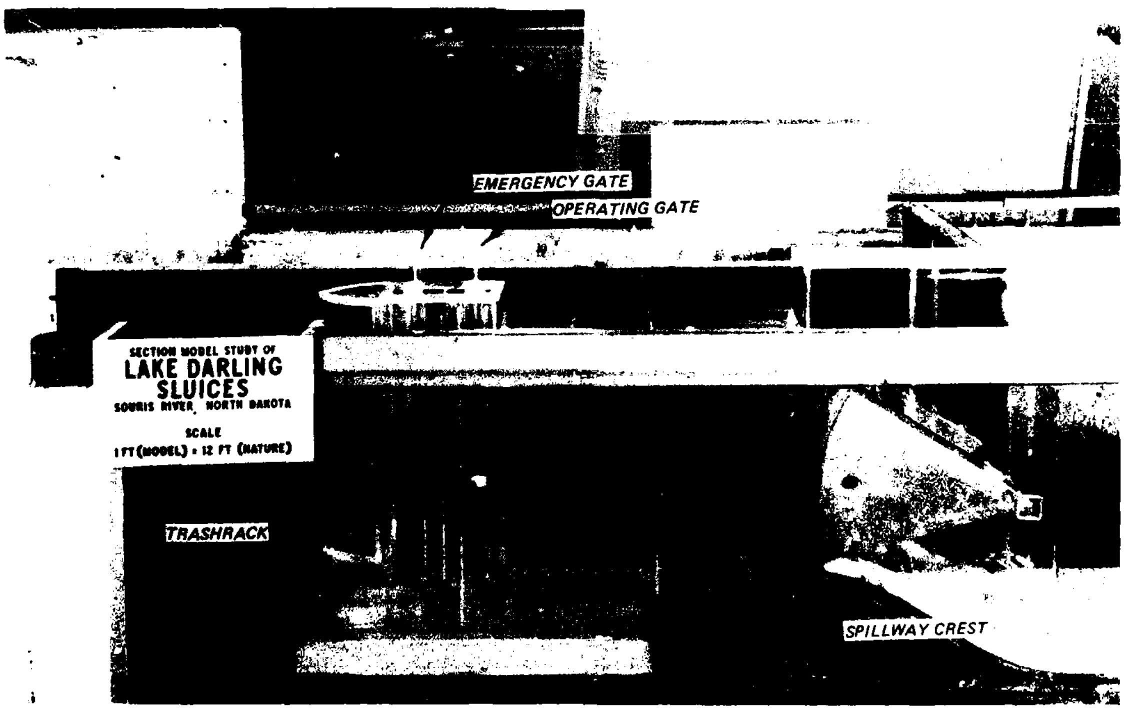


Figure 3. The 1:12-scale sluice model

Figure 4. Montedoro Whitney model MVM-1 velocity meter

relations for the transference of model data to prototype equivalents are presented in the following tabulation:

| | | Scale Relations | |
|---|---|---|---|
| Dimension* | Ratio | 1:36-Scale Model | 1:12-Scale Model |
| Length | $L_r = L$ | 1:36 | 1:12 |
| Area | $A_r = L_r^2$ | 1:1,296 | 1:144 |
| Weight | $W_r = L_r^3$ | 1:46,656 | 1:1,728 |
| Velocity | $V_r = L_r^{1/2}$ | 1:6 | 1:3.464 |
| Discharge | $Q_r = L_r^{5/2}$ | 1:7,776 | 1:498.831 |
| Time | $T_r = L_r^{1/2}$ | 1:6 | 1:3.464 |

* Dimensions are in terms of length.

11. Quantitative measurements of discharge, water-surface elevation, and velocity in the model were converted to prototype dimensions by means of these scale relations. Experimental data also indicate that the prototype-to-model scale ratio is valid for scaling riprap in the sizes used in this investigation.

## PART III: TESTS AND RESULTS

### Approach Area

#### Original design

12. While the general pattern of flow approached the spillway at a slight angle, flow conditions in the spillway approach were generally satisfactory except in the immediate vicinity of the right wing wall and upstream left bank (Figures 5 and 6) for discharges of 45,000 cfs and greater. Surface eddies formed along the left bank and dissipated in the channel upstream of the spillway (Photos 1 and 2). Flow approaching the spillway from the right contracted along the right abutment, a condition which became progressively severer as the discharge was increased. Flow was evenly distributed across the spillway, except in the right bay along the inside of the approach wall. Isovels and water-surface profiles along the weir axis at sta 0+49 are shown in Plate 4 for the design discharge of 99,800 cfs.

#### Alternate designs

13. During the model investigation, the left bank approach was modified in the prototype. A 30-ft-wide horizontal bench was added at el 1,601.0 along the approach, and the slope above the bench was changed to 1V on 4H (Figure 7 and Plate 5). This approach (type 2) produced a large eddy (Figure 8) that caused flow to be constricted in the channel. This constriction resulted in a higher pool elevation for discharges above 50,000 cfs than was produced with the original approach configuration (type 1), as discussed in paragraph 16.

14. Although the vorticity and turbulence observed in the vicinity of the approach wall were considered satisfactory with the original design abutment, remedial measures to improve flow conditions in these areas were investigated with the model. These measures involved elliptical (type 2) and parabolic (type 3) walls with 7-ft-radius noses at the right abutment (Plate 6), or a 7-ft-radius nose (type 4) at the end of the original design wall (Plate 7, Photos 3 and 4). The maximum water-surface differential between the back and inside of the type 1 (original design) wall occurred at the design discharge (99,800 cfs) at section A-A as shown in Plate 8. Water-surface differentials for drawdown with the types 3 and 4 design walls installed in the model are shown in Plates 9 and 10, respectively. Observations revealed that while each of these designs improved flow conditions,

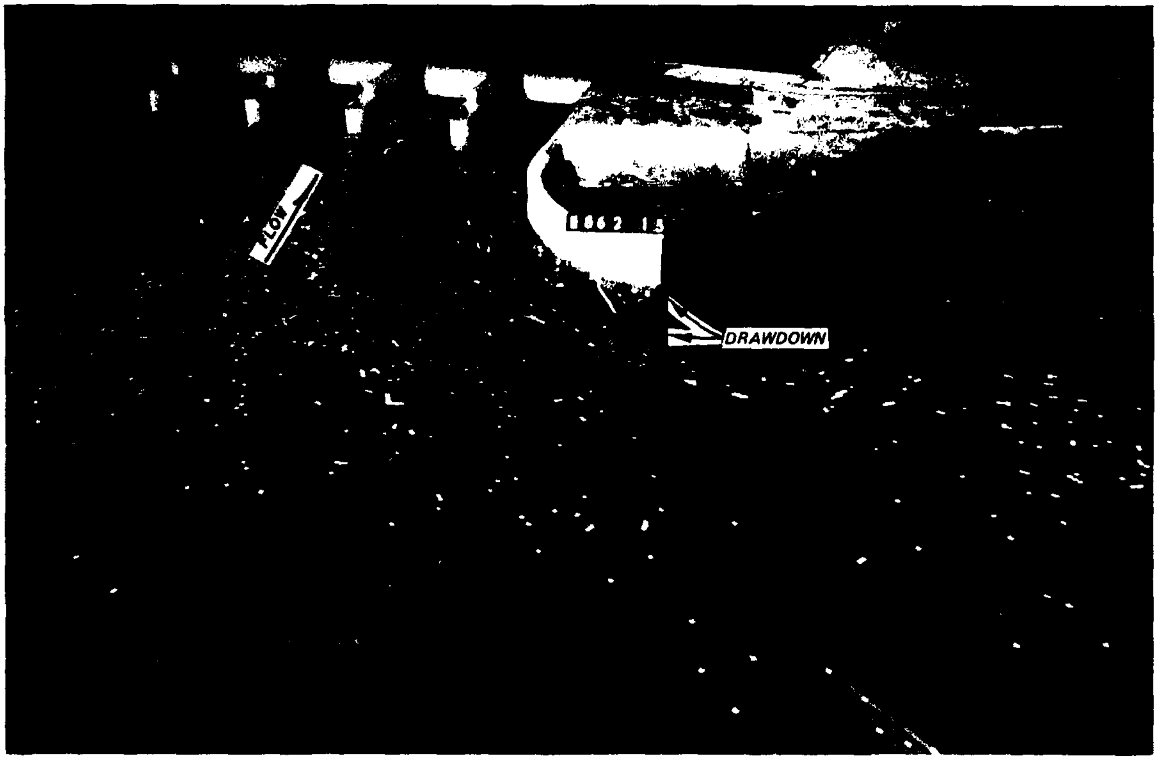


Figure 5. Right approach wall; white streaks (confetti), surface currents; type 1 (original) left bank approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates open full


Figure 6. Upstream approach; type 1 (original) left bank; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates full open

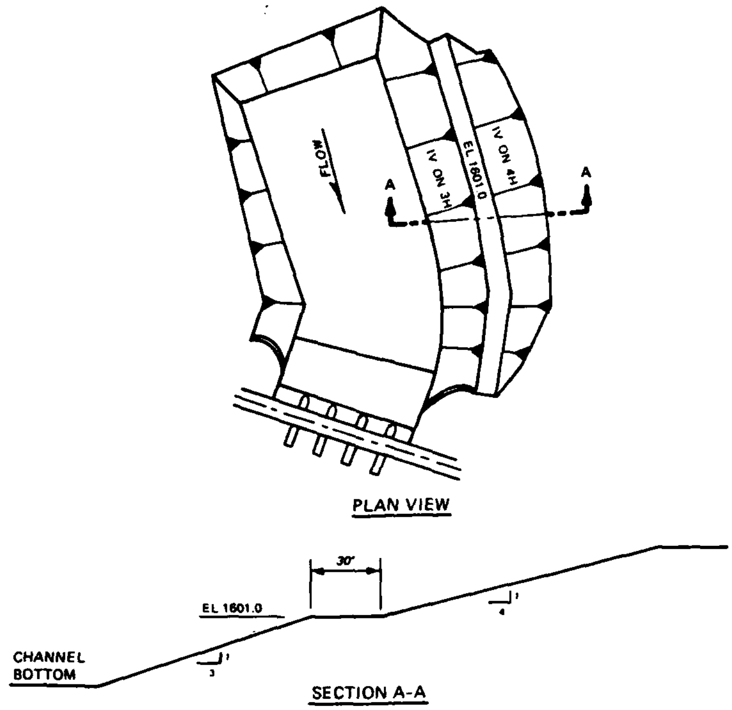


Figure 7. Plan and profile views, type 2 upstream approach


Figure 8. Type 2 upstream approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates open full

particularly at the higher flows, the slight degree of improvement obtained did not justify the additional cost of these walls. The alternate design walls had little or no effect on the discharge capacity of the spillway. Tests of these various right approach wall shapes resulted in the recommendation of the type 1 (original) design right wall for prototype construction (Figure 9).

Figure 9. Type 1 (original) right wall and approach

## Crest and Piers

15. Details of the spillway crest and piers are shown in Plates 2 and 3. The weir was 12.5 ft high with a 1-on-1 sloping upstream face with an ogee crest. The 10-ft-thick crest piers extended 58 ft upstream from the weir crest. The piers and sluices terminated at the downstream toe of the crest. The 3- by 4-ft sluices extended through the length of the pier as shown. During the model tests, no changes were made to the spillway weir or piers.

## Spillway Capacity

16. As mentioned, the type 2 left bank approach produced a large eddy along the left bank (Figure 8) that caused flow to be constricted in the channel. As a result, the discharge capacity of the spillway was reduced for free uncontrolled flow with discharges greater than 50,000 cfs. Spillway rating curves for all five gates at full and partial openings for the type 1 (original) and type 2 left bank approaches are shown in Plates 11 and 12, respectively, for free flow conditions. The basic calibration data are shown in plots of the approach channel energy elevation (water surface plus velocity head based on average velocity) on the weir versus discharge for free flows at full and partial gate openings for the types 1 and 2 left bank approaches, respectively, in Plates 13-16. Data used to plot these curves are shown in Table 1. These curves were obtained by introducing several constant discharges into the model for each gate opening and recording the corresponding upper pool elevation for minimum tailwater conditions. The equation for each of these curves is the best empirical fit of the free-flow data by the method of least squares. The efficiency of the structure was slightly reduced by modifying the original left bank approach. The type 2 left bank approach has been constructed in the prototype and will remain in existence after completion of construction of the dam. The maximum pool elevation (el 1,608.5) with the type 2 left bank approach was less than for the original design (el 1,609) at the design flood (99,800 cfs). This indicated that the overall design of the structure is slightly more efficient than the original design.

17. Tailwater submergence effects on the coefficient of discharge were determined in the model at 99,800 cfs as requested by the Office, Chief of Engineers, US Army. Model data were compared to data plotted on Hydraulic Design Chart 111-4.* The data indicated that the submergence effects of the tailwater on the coefficient of discharge were negligible (Plate 17).

## Spillway Crest Pressures

18. Spillway crest pressures were investigated in the model. Minimum

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* US Army Corps of Engineers. "Submerged Crest Coefficients, Overflow Crests," Hydraulic Design Chart 111-4, Hydraulic Design Criteria, prepared for Office, Chief of Engineers, by US Army Engineer Waterways Experiment Station, Vicksburg, Miss., issued serially since 1952.

pressures of 4.7 and 2.3 ft of water, respectively, occurred along the center of the gate bay and along the crest piers during release of the spillway design discharge. Negative pressures of 2.8 and 1.1 ft of water, respectively, occurred along the center of the gate bay and along the crest piers during controlled release of 5,000 cfs (one gate open 2.0 ft). With pressures of these magnitudes, cavitation along the weir crest should not be a problem.

19. No unstable or periodic surging occurred in the gate bays upstream from the gates during controlled operation of the spillway.

## Stilling Basin

### Type 1 (original design)

20. The original stilling basin (Figure 10, Plate 3) consisted of an 80-ft-long apron at el 1,571.5 with two rows of 4.5-ft-high baffle piers and a 2.2-ft-high sloped end sill. Sidewalls were vertical and were 29.8 ft high (top el 1,601.3). The wing walls adjacent to the stilling basin end sill were battered at a 10V on 1H slope (Plate 1). With the basin at el 1,571.5, natural tailwater provided 100 percent of the theoretical depth, $D_2$ , required for

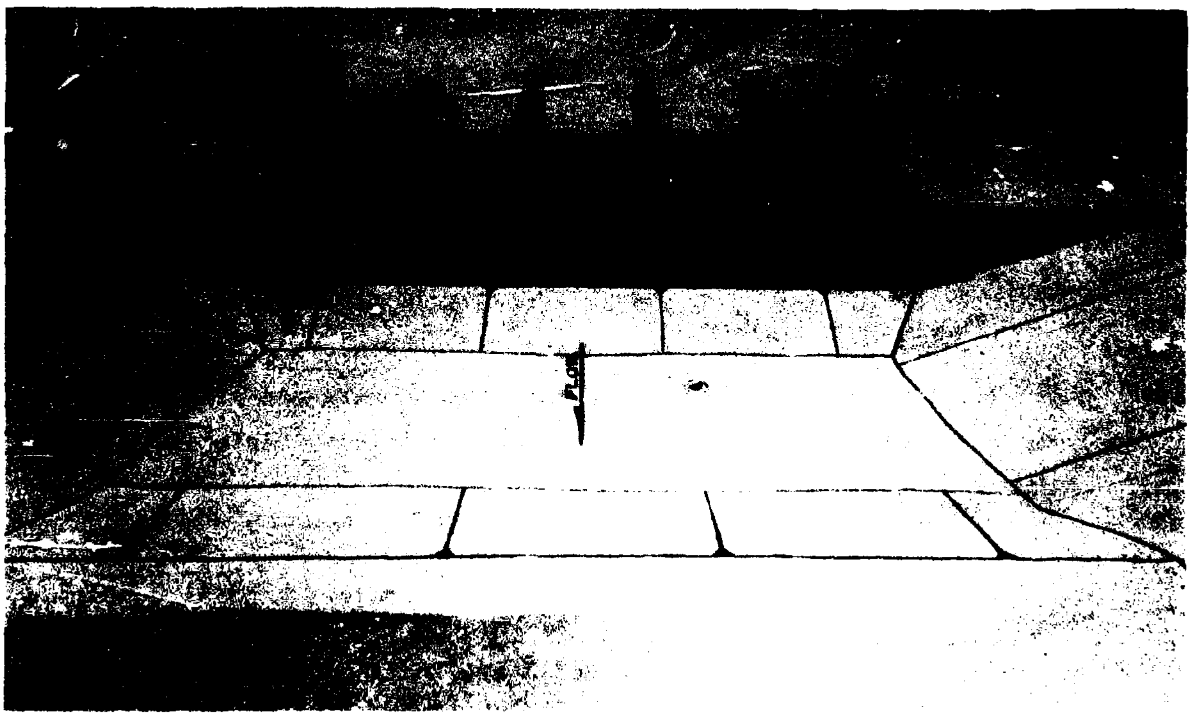


Figure 10. Type 1 (original) stilling basin

a hydraulic jump for the range of discharges up to 99,800 cfs (PMF). Tailwater elevations were set according to the expected tailwater curve shown in Plate 18.

21. Observation of flow conditions in the original design basin revealed an oscillating jump in the basin with resulting wave action in the exit area. This action was anticipated due to the low Froude number (2.3) of entering flow. The wave action in the exit was further amplified by the preformed scour hole immediately downstream of the stilling basin (Figure 10). The preformed scour hole was removed before testing continued.

22. A water-surface profile with the original design basin for the design discharge of 99,800 cfs (Photo 5) is shown in Plate 19. Maximum bottom velocities along the center line of the stilling basin varied from 14 to 26 fps for the design discharge. Bottom velocities in the stilling basin are shown in Plate 20.

Alternate designs

23. Energy dissipation in the original basin was not considered adequate because of severe damage to the riprap immediately downstream of the end sill after 1 hr at the PMF of 99,800 cfs. Alternate stilling basin designs, in combination with various riprap schemes, were examined in an effort to design cost-effective protection for the downstream channel as requested by the sponsor.

24. The type 1 basin was modified to the type 2 basin that consisted of a 106-ft-long apron at el 1,571.5 with two rows of 6-ft-high baffle piers and a 4.0-ft-high sloped end sill (Plate 21). The baffle blocks were sized for the design discharge of 99,800 cfs. The types 3-5 basins were similar in length and differed only in the size and location of the basin elements (Plate 22).

25. Observations of the type 2 basin revealed that unsatisfactory energy dissipation occurred for a discharge of 99,800 cfs (Photo 6). Maximum bottom velocities along the center line of the stilling basin varied from 4 to 24 fps for the design discharge. Bottom velocities are shown in Plate 23. A water-surface profile with the type 2 basin for the design discharge of 99,800 cfs is shown in Plate 24.

26. The alternate designs did not improve downstream flow conditions from the standpoint of hydraulic performance at the design discharge. Large standing waves were produced at the discharge of 99,800 cfs with the type 2

basin. No further attempts were made to develop a better stilling basin design for two reasons: (a) the longer basin with larger baffle blocks did not provide enough improvement in energy dissipation to warrant the additional cost of prototype construction; and (b) the basin rests on a strata of bedrock that prevents lowering the basin. The original basin performed adequately for flows up to the standard project flood of 45,000 cfs. Therefore, the original design stilling basin (Figure 10) will be used for prototype construction.

27. The stilling basin training walls were of adequate height to prevent overtopping at all discharges.

## Sluices

28. The 1:12-scale section model (Figure 3) of the sluice was used to study discharge characteristics, flow conditions, and pressures in the sluice. To measure pressures along the roof of the sluice, 40 piezometer tubes were placed along the sluice center line at locations shown in Plate 25.

29. Initial tests were conducted to determine the relationship between headwater and tailwater elevations for sluice gate openings of 1.0, 2.0, 3.0, and 4.0 ft with discharges of 100, 200, 300, and 400 cfs. The headwater water-surface elevation was measured using a point gage located 30 ft upstream of the sluice inlet. Headwater and tailwater curves for various gate openings and discharges are plotted in Plates 26-29.

30. The model was observed with pool elevations ranging from 1,591 ft to 1,605 ft, sluice gate openings ranging from 1 to 4 ft (full), and tailwater elevations from 1,573 ft (minimum) to 1,600 ft to determine flow conditions and pressures in the sluice. Three types of flow conditions occurred in the conduit downstream from the gate: (a) open channel flow (free water surface); (b) slug flow (pockets of air along the top of the sluice moving with flow); and (c) full pressure flow. These flow conditions were dependent on pool elevation, tailwater elevation, and gate opening. A tabulation of the type of flow condition observed with various combinations of pool and tailwater elevations for gate openings of 1.0, 2.0, 3.0, and 4.0 ft are shown in Tables 2, 3, 4, and 5 respectively. These data are also shown in Plates 26-29.

31. Tests were conducted as requested by the sponsor to determine the pool elevation at which vortices would form above the sluice intake with

minimum tailwater and full gate opening. Vortices formed when the pool was lowered to el 1,584.

32. Pressure data were recorded for minimum tailwater (el 1,573) and minimum pool (el 1,596); minimum tailwater (el 1,573) and maximum pool (el 1,605); and maximum tailwater (el 1,598.3) and maximum pool (el 1,605) conditions with gate openings of 1.0, 2.0, 3.0, and 4.0 ft (Table 6). At gate openings of 1.0-3.0 ft, open channel flow occurred downstream with minimum tailwater for both minimum and maximum pool conditions. Pressures upstream of the gate were recorded. No negative pressures were observed downstream from the gate.

## Riprap Requirements and Exit Channel Configuration

### Upstream

33. Details of the riprap protection in the approach area and along the embankment as tested in the 1:36-scale general model are shown in Plate 30. For all tests, the approach area and embankment were covered with protective stone simulating prototype stone with an average weight of 26 lb. Riprap gradation curves for all riprap used in testing are plotted in Plates 31-33. The upstream and embankment riprap protection remained stable during all discharges. Thus, attention was focused on improving downstream flow conditions and downstream channel bottom riprap stability.

### Downstream

34. Proposed riprap protection was tested with the original (80-ft-long) and type 2 (106-ft-long) basins in an effort to determine the most economical combination of stilling basin and riprap protection design as requested by the sponsor.

35. Return flow (Plate 34) on either side of the downstream channel was observed in the model due to the flare of the exit channel as originally designed. The return flow was alleviated by constricting the outlet channel bottom width to 265 ft (Plate 35, Figure 11). The type 1 (original) design riprap protection was placed in the downstream channel and along the berm as shown in Plate 36. For a distance of 206 ft downstream of the end sill of the type 1 stilling basin (sta 1+14 to 3+20), the channel bottom and side slopes were covered with a 36-in.-thick blanket of protective stone simulating prototype stone with an average weight of 207 lb. For 80 ft (sta 3+20 to 4+00),


Figure 11. Modified exit channel, type 1 (original) left bank approach

the channel bottom and side slopes were covered with a 24-in.-thick blanket of protective stone simulating prototype stone with an average weight of 79 lb. For 500 ft (sta 4+00 to 9+00) the channel bottom and side slopes were covered with a 12-in.-thick blanket of protective stone simulating prototype stone with an average weight of 26 lb (Plate 37). The type 1 (original) riprap design was modified as shown in Plate 38 for use with the longer (type 2) basin. Initial scour tests conducted with the type 1 basin (80 ft long) revealed that the riprap was stable for discharges less than 45,000 cfs. Initial riprap failure at 45,000 cfs occurred on the left side of the channel and progressed downstream to the right in the channel bottom. After 18 hr at the design discharge, 100 ft of the riprap was scoured as shown in Plate 37. With the longer type 2 basin (106 ft long), the riprap remained stable for discharges less than 90,000 cfs. After 18 hr at the design discharge, a 73-ft length of stone on the left of the structure scoured as shown in Plate 38. Riprap protection on the berm remained stable for the full range of discharges.

36. The original riprap gradation was grouted for 206 ft in the type 2 design riprap protection (Plate 39) for the shorter (type 1) basin. This protection plan was stable for all discharges. Doubling the thickness of the original riprap protection to 72 in. (type 3 riprap plan) did not improve riprap stability at the design discharge. The type 4 design riprap plan (Plate 39) for use with the type 2 stilling basin was similar in gradation to the type 2 riprap design and differed only in the length of the grouted section. The grouted protection and riprap remained stable for the full range of discharges.

37. Tests were conducted to determine a minimum size of ungrouted riprap protection that would remain stable for the full range of discharges for both stilling basins. Types 5 and 6 riprap designs (Plate 40) were tested in conjunction with the longer (type 2) basin. Types 7 and 8 riprap designs (Plate 41) were tested in conjunction with the original (type 1) basin.

38. The type 5 riprap design consisted of a 94-ft-long, 54-in.-thick blanket of uniformly graded protective stone simulating prototype stone with an average weight of 984 lb, followed by a 122-ft-long, 36-in.-thick blanket of protective stone with an average weight of 207 lb, and a 544-ft-long, 12-in.-thick blanket of protective stone with an average weight of 26 lb. The gradation remained the same with the type 6 design, but the length of each blanket differed as shown in Plate 40. The type 5 riprap remained stable at the design discharge while there was displacement of the type 6 riprap design at the same discharge.

39. The type 7 riprap design consisted of a 94-ft-long, 54-in.-thick blanket of uniformly graded protective stone with an average weight of 984 lb, followed by a 147-ft-long, 36-in.-thick blanket of protective stone with an average weight of 207 lb, and a 544-ft-long, 12-in.-thick blanket of protective stone with an average weight of 26 lb. The riprap 100 ft downstream of the end sill failed at the design discharge. The type 8 design was similar in gradation to the type 7 and differed only in the length of the 54-in.- and 36-in.-thick blankets as shown in Plate 41. This design remained stable for the full range of discharges.

40. Grouting the riprap for a certain distance immediately downstream from the stilling basin will provide protection of either basin for the design discharge of 99,800 cfs. For protection of the type 1 basin, riprap with an average size stone of 984 lb should be used for a distance of 120 ft

downstream from the end sill; stone weighing 207 lb can be used to transition into a 26-lb stone throughout the remainder of the exit channel (type 8 design riprap). For protection of the type 2 basin, riprap with an average size stone of 984 lb should be used for a distance of 94 ft downstream from the end sill; stone weighing 207 lb can be used to transition into a 26-lb stone throughout the remainder of the exit channel (type 5 design riprap).

## PART IV: DISCUSSION AND RECOMMENDATIONS

41. Performance of the approach channel of original design was generally satisfactory. Although some surface vorticity in the vicinity of the abutment walls was observed, the use of elliptical or parabolic walls to alleviate this condition was not warranted either economically or hydraulically.

42. During the model investigation, the left bank approach was modified in the prototype. This modification reduced the discharge efficiency of the structure with the larger flows, but the structure will still pass the design flow at the pool elevation originally anticipated. Equations were developed to compute discharge through the structure with free-controlled, and uncontrolled flows. The effects of tailwater submergence on discharge with the tailwaters expected at this project were negligible.

43. Alternate stilling basin designs did not significantly improve downstream flow conditions from the standpoint of hydraulic performance at the design discharge (99,800 cfs). The original 80-ft-long stilling basin performed adequately for discharges up to 45,000 cfs. Although the jump held in the original basin with discharges of 46,000 cfs and higher, the riprap downstream of the end sill failed with a discharge of 45,000 cfs. Bedrock prevented the lowering of the basin. Therefore, the original basin was recommended for prototype construction, and a downstream protection plan was developed to prevent scour.

44. Observations indicated that average size stone of 207 lb should be used for a distance of at least 206 ft downstream of the stilling basin and that stone weighing 79 lb would be sufficient for protection of the remainder of the exit channel for flows up to 45,000 cfs. Average size stone of 984 lb should be used for a distance of at least 120 ft if protection is desired with the design discharge (PMF) of 99,800 cfs. Upstream of the structure, the stone size selected for wave protection along the embankment would be sufficiently large to protect against velocities at the spillway abutments.

45. Flow conditions in the sluices were generally satisfactory for the various discharge and tailwater combinations. No negative pressures occurred. Slug flow could occur with some discharge-tailwater combinations.

Table 1

Data for Plotting Rating Curves

| | Type 1 Approach | | | Type 2 Approach | | |
|---|---|---|---|---|---|---|
| Q | WS El | $H_g$ | $H_g + \frac{\bar{V}^2}{2g}$ | WS El | $H_g$ | $H_g + \frac{\bar{V}^2}{2g}$ |
| | | | $G_o$ = 5.0 ft | | | |
| 21,000 | 1,597.1 | 10.6 | 10.6 | 1,597.1 | 10.6 | 10.6 |
| 22,500 | 1,598.8 | 12.3 | 12.3 | 1,598.8 | 12.3 | 12.3 |
| 25,000 | 1,602.2 | 15.7 | 15.7 | 1,602.1 | 15.6 | 15.6 |
| 27,500 | 1,605.6 | 19.1 | 19.1 | 1,605.9 | 19.4 | 19.4 |
| 30,000 | 1,609.4 | 22.9 | 22.9 | 1,609.3 | 22.8 | 22.8 |
| | | | $G_o$ = 10.0 ft | | | |
| 47,500 | 1,601.2 | 12.2 | 12.4 | 1,602.6 | 13.6 | 13.7 |
| 50,000 | 1,602.7 | 13.7 | 13.9 | 1,605.1 | 16.1 | 16.3 |
| 52,500 | 1,604.2 | 15.2 | 15.4 | 1,606.5 | 17.5 | 17.7 |
| 55,000 | 1,605.9 | 16.9 | 17.1 | 1,608.5 | 19.5 | 19.7 |
| 57,500 | 1,607.7 | 18.7 | 18.8 | 1,610.3 | 21.3 | 21.5 |
| 59,700 | 1,609.4 | 20.4 | 20.6 | -- | -- | -- |
| | | | $G_o$ = 15.0 ft | | | |
| 80,000 | 1,606.5 | 15.0 | 15.3 | 1,608.9 | 17.4 | 17.7 |
| 82,500 | 1,607.5 | 16.0 | 16.3 | 1,610.7 | 19.2 | 19.5 |
| 85,000 | 1,608.5 | 17.0 | 17.3 | 1,611.9 | 20.4 | 20.7 |
| 87,500 | 1,609.5 | 18.0 | 18.2 | 1,612.0 | 20.5 | 20.8 |

(Continued)

Note: Q = discharge, cfs
WS = water-surface elevation, ft NGVD
$H_g$ = head on the gate, ft
$H_g + \frac{\bar{V}^2}{2g}$ = head on the gate + velocity head, ft
$G_o$ = gate opening
H = head on the crest, ft
$H + \frac{\bar{V}^2}{2g}$ = head on the crest + velocity head, ft

Table 1 (Concluded)

| | Type 1 Approach | | | Type 2 Approach | | |
|---|---|---|---|---|---|---|
| Q | WS El | $H_g$ | $H_g + \frac{\overline{V}^2}{2g}$ | WS El | $H_g$ | $H_g + \frac{\overline{V}^2}{2g}$ |
| | | $G_o = \infty$ Uncontrolled | | | | |
| 30,000 | 1,594.8 | 10.8 | 10.8 | 1,594.8 | 10.8 | 10.8 |
| 40,000 | 1,596.9 | 12.9 | 13.2 | 1,596.9 | 12.9 | 13.2 |
| 50,000 | 1,599.0 | 15.0 | 15.4 | 1,599.0 | 15.0 | 15.4 |
| 55,000 | 1,600.2 | 16.2 | 16.7 | 1,600.2 | 16.2 | 16.7 |
| 57,500 | 1,600.5 | 16.5 | 16.9 | 1,600.5 | 16.5 | 16.9 |
| 60,000 | 1,601.0 | 17.0 | 17.4 | 1,602.0 | 18.0 | 18.2 |
| 70,000 | 1,602.4 | 18.4 | 18.8 | 1,603.7 | 19.7 | 20.0 |
| 80,000 | 1,604.4 | 20.4 | 20.9 | 1,605.5 | 21.5 | 22.0 |
| 90,000 | 1,606.0 | 22.0 | 22.6 | 1,606.4 | 22.4 | 23.0 |
| 99,800 | 1,607.8 | 23.8 | 24.4 | 1,608.5 | 24.5 | 25.1 |
| 100,000 | 1,608.2 | 24.2 | 24.8 | 1,608.5 | 24.5 | 25.1 |
| 110,000 | 1,608.8 | 24.8 | 25.4 | 1,609.9 | 25.9 | 26.5 |
| 120,000 | 1,610.2 | 26.2 | 26.9 | 1,611.5 | 27.5 | 28.2 |
| 125,000 | 1,611.0 | 27.0 | 27.7 | 1,612.4 | 28.4 | 29.1 |

Table 2

Lake Darling Sluice Flow Conditions

$G_o = 1.0$ ft

| Tailwater El* | Flow Condition for Pool El 1,591 | 1,592 | 1,593 | 1,594 | 1,595 | 1,596 | 1,597 | 1,598 | 1,599 | 1,600 |
|---|---|---|---|---|---|---|---|---|---|---|
| 1,574 | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel |
| 1,575 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,576 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,577 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,578 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,580 | Slug | Slug | Slug | Slug | Slug | Slug | | | | |
| 1,582 | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Slug | Slug | Slug | Slug |
| 1,584 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | Full Pressure | Full Pressure | Full Pressure | Full Pressure |
| 1,586 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,590 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,595 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |
| 1,600 | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ | ↓ |

* All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Table 3

Lake Darling Sluice Flow Conditions

$G_o$ = 2.0 ft

| Tailwater El* | Flow Condition for Pool El | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1,591 | 1,592 | 1,593 | 1,594 | 1,595 | 1,596 | 1,597 | 1,598 | 1,599 | 1,600 |
| 1,574 | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel |
| 1,575 | | | | | | | | | | |
| 1,576 | | | | | | | | | | |
| 1,577 | | | | | | | | | | |
| 1,578 | | | | | | | | | | |
| 1,580 | | | | | | | | | | |
| 1,582 | Slug | Slug | Slug | Slug | | | | | | |
| 1,584 | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Slug | Slug | Slug | Slug | Slug | Slug |
| 1,586 | | | | | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure |
| 1,590 | | | | | | | | | | |
| 1,595 | | | | | | | | | | |
| 1,600 | | | | | | | | | | |

* All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Table 4

Lake Darling Sluice Flow Conditions

$G_o$ = 3.0 ft

| Tailwater El | Flow Condition for Pool El | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1,591 | 1,592 | 1,593 | 1,594 | 1,595 | 1,596 | 1,597 | 1,598 | 1,599 | 1,600 |
| 1,574 | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel | Open Channel |
| 1,575 | | | | | | | | | | |
| 1,576 | | | | | | | | | | |
| 1,577 | | | | | | | | | | |
| 1,578 | | | | | | | | | | |
| 1,580 | | | | | | | | | | |
| 1,582 | Slug | Slug | Slug | Slug | | | | | | |
| 1,584 | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Slug | Slug | Slug | Slug | Slug | |
| 1,586 | | | | | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Slug |
| 1,590 | | | | | | | | | | Full Pressure |
| 1,595 | | | | | | | | | | |
| 1,600 | | | | | | | | | | |

* All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Table 5

Lake Darling Sluice Flow Conditions

$G_o = 4.0$ ft

| Tailwater El | Flow Condition for Pool El | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1,591 | 1,592 | 1,593 | 1,594 | 1,595 | 1,596 | 1,597 | 1,598 | 1,599 | 1,600 |
| 1,574 | Slug | Slug | Slug | Slug | Slug | Slug | Slug | Slug | Slug | Slug |
| 1,575 | | | | | Slug | Slug | Slug | Slug | Slug | Slug |
| 1,576 | | | | | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air |
| 1,577 | | | | | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air | Pressure w/Air |
| 1,578 | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure | Full Pressure |
| 1,580 | | | | | | | | | | |
| 1,582 | | | | | | | | | | |
| 1,584 | | | | | | | | | | |
| 1,586 | | | | | | | | | | |
| 1,590 | | | | | | | | | | |
| 1,595 | | | | | | | | | | |
| 1,600 | | | | | | | | | | |

* All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

Table 6

Lake Darling Sluices

Pressures in ft of Water

| Piezometer No. | Piezometer EL | Gate Opening, ft, at Tailwater El 1,573.0 — Pool El 1596: 1 | 2 | 3 | 4 | Pool El 1605: 1 | 2 | 3 | 4 | Gate Opening, ft, at Tailwater El 1,598.3 — Pool El 1605: 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1,578.8 | 18.5 | 18.2 | 18.4 | 17.4 | 26.5 | 27.0 | 27.0 | 26.1 | 26.9 | 27.0 | 27.5 | 26.8 |
| 2 | 1,578.3 | 18.5 | 18.5 | 15.9 | 12.2 | 26.7 | 25.9 | 23.4 | 19.0 | 27.4 | 27.4 | 27.4 | 25.5 |
| 3 | 1,577.9 | 18.8 | 18.7 | 15.8 | 11.7 | 26.9 | 25.8 | 23.4 | 17.8 | 27.7 | 27.3 | 26.6 | 25.5 |
| 4 | 1,577.7 | 19.0 | 18.7 | 15.8 | 11.3 | 27.1 | 25.9 | 23.0 | 16.9 | 27.9 | 27.3 | 26.7 | 25.5 |
| 5 | 1,577.5 | 19.1 | 19.2 | 15.0 | 9.2 | 27.2 | 25.3 | 21.5 | 13.3 | 28.0 | 27.3 | 26.2 | 25.0 |
| 6 | 1,577.3 | 18.9 | 18.1 | 13.9 | 7.2 | 26.9 | 24.6 | 20.0 | 10.2 | 28.1 | 27.3 | 25.9 | 24.4 |
| 7 | 1,577.2 | 19.3 | 17.5 | 12.8 | 5.3 | 27.1 | 24.4 | 19.1 | 8.1 | 28.2 | 27.2 | 25.4 | 23.9 |
| 8 | 1,577.1 | 19.3 | 17.4 | 12.5 | 4.4 | 27.0 | 24.2 | 18.5 | 6.6 | 28.3 | 27.1 | 25.2 | 23.6 |
| 9 | 1,577.0 | 19.5 | 17.2 | 12.0 | 3.4 | 27.1 | 24.0 | 27.8 | 5.3 | 28.4 | 27.1 | 25.2 | 23.6 |
| 10 | 1,577.0 | 19.4 | 17.1 | 11.7 | 3.2 | 26.9 | 24.8 | 17.5 | 4.6 | 28.4 | 27.0 | 24.8 | 23.3 |
| 11 | 1,577.0 | 19.7 | 17.3 | 12.1 | 3.5 | 26.5 | 24.7 | 17.6 | 4.9 | 28.4 | 27.0 | 24.9 | 23.4 |
| 12 | 1,577.0 | 19.4 | 17.3 | 12.3 | 4.1 | 27.2 | 24.2 | 18.2 | 5.8 | 28.4 | 27.1 | 25.2 | 23.7 |
| 13 | 1,577.0 | 19.5 | 17.3 | 12.2 | 4.0 | 27.2 | 24.2 | 18.0 | 5.6 | 28.4 | 27.1 | 25.0 | 23.6 |
| 14 | 1,577.0 | 19.4 | 17.4 | 11.8 | 2.8 | 26.9 | 23.7 | 17.4 | 3.7 | 28.4 | 27.0 | 24.7 | 23.2 |
| 15 | 1,577.0 | 18.7 | 16.6 | 11.7 | 3.7 | 26.5 | 23.5 | 17.0 | 4.6 | 28.4 | 27.1 | 24.8 | 23.2 |
| 16 | 1,577.0 | | | | | | | | | 28.4 | 17.5 | 17.7 | 23.1 |
| 17 | 1,577.0 | | | | | | | | | 28.4 | 17.5 | 18.6 | 22.8 |
| 18 | 1,577.0 | | | | | | | | | 19.4 | 17.8 | 20.9 | 22.7 |
| 19 | 1,577.0 | | | | | | | | | 19.4 | 18.7 | 20.8 | 22.6 |
| 20 | 1,577.0 | | | | | | | | | 19.3 | 19.8 | 21.0 | 22.6 |
| 21 | 1,577.0 | | | | | | | | | 19.6 | 20.5 | 22.0 | 22.5 |
| 22 | 1,577.0 | | | | | | | | | 19.9 | 21.4 | 22.1 | 22.4 |
| 23 | 1,577.0 | | | | | | | | | 20.5 | 21.8 | 22.1 | 22.3 |
| 24 | 1,577.0 | | | | | | | | | 21.0 | 21.9 | 22.1 | 22.2 |
| 25 | 1,577.0 | | | | | | | | | 21.7 | 21.9 | 22.0 | 22.1 |
| 26 | 1,577.0 | | | | | | | | | 21.7 | 21.9 | 22.0 | 22.0 |
| 27 | 1,577.0 | | | | | | | | | 21.7 | 21.9 | 21.8 | 21.8 |
| 28 | 1,577.0 | | | | | | | | | 21.7 | 21.9 | 21.8 | 21.8 |
| 29 | 1,577.0 | | | | | | | | | 21.7 | 21.8 | 21.8 | 21.7 |
| 30 | 1,577.0 | | | | | | | | | 21.7 | 21.8 | 21.7 | 21.6 |
| 31 | 1,577.0 | | | | | | | | | 21.7 | 21.8 | 21.6 | 21.5 |
| 32 | 1,577.0 | | | | | | | | | 21.7 | 21.8 | 21.6 | 21.5 |
| 33 | 1,577.0 | | | | | | | | | 21.7 | 21.7 | 21.6 | 21.5 |
| 34 | 1,577.0 | | | | | | | | | 21.7 | 21.8 | 21.6 | 21.7 |
| 35 | 1,577.0 | | | | | | | | | 21.7 | 21.7 | 21.5 | 21.3 |
| 36 | 1,577.0 | | | | | | | | | 21.7 | 21.7 | 21.5 | 21.5 |
| 37 | 1,577.0 | | | | | | | | | 21.8 | 21.8 | 21.9 | 22.0 |
| 38 | 1,576.9 | | | | | | | | | 21.9 | 22.1 | 22.4 | 22.5 |
| 39 | 1,576.9 | | | | | | | | | 22.0 | 22.3 | 22.7 | 22.8 |
| 40 | 1,576.7 | | | | | | | | | 22.2 | 22.7 | 23.0 | 23.1 |

Note: On tests 16-40, no piezometer readings were recorded for pool el 1,596 and 1,605 because tailwater elevations were below piezometer levels.

Photo 1. Type 4 design right wall; type 1 (original) left bank approach; discharge 45,000 cfs; pool el 1,605.0; tailwater el 1,593.0; gate opening 10.0 ft

Photo 2. Type 4 design right wall; type 1 (original) left bank approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates fully open

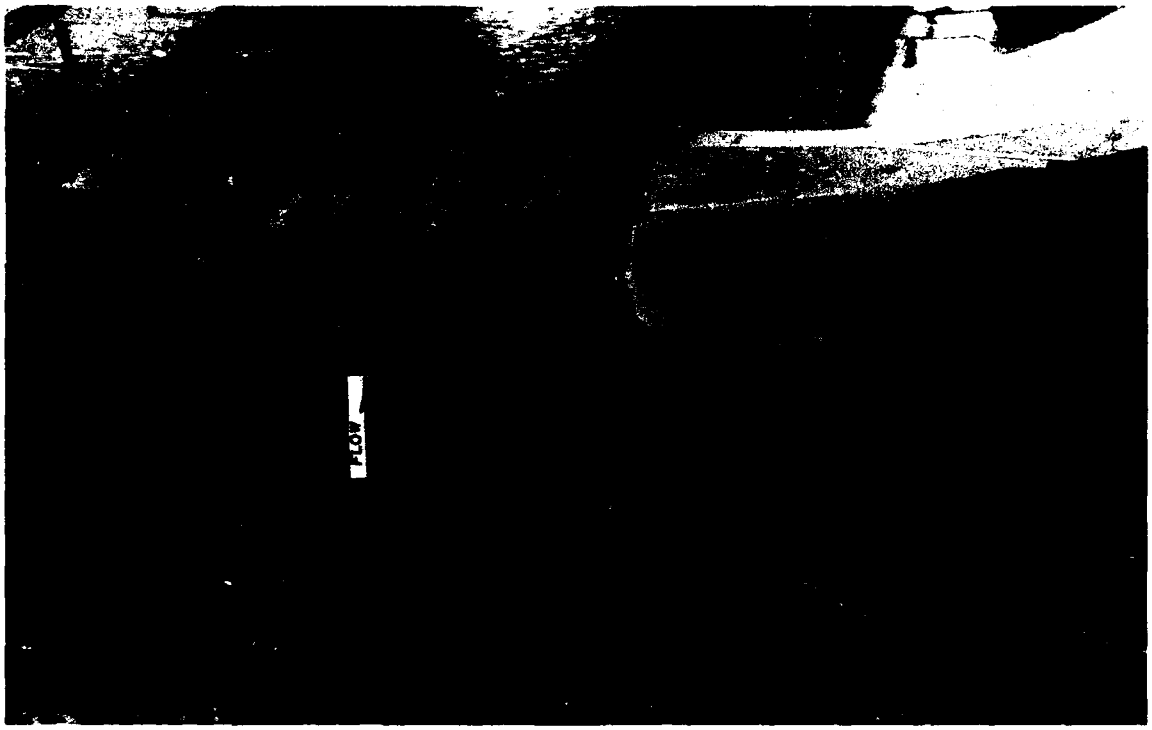


Photo 3. Type 4 design right wall; type 1 (original) left bank approach; discharge 45,000 cfs; pool el 1,605.0; tailwater el 1,593.0; gate opening 10.0 ft

Photo 4. Type 4 design right wall; type 1 (original) left bank approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates fully open

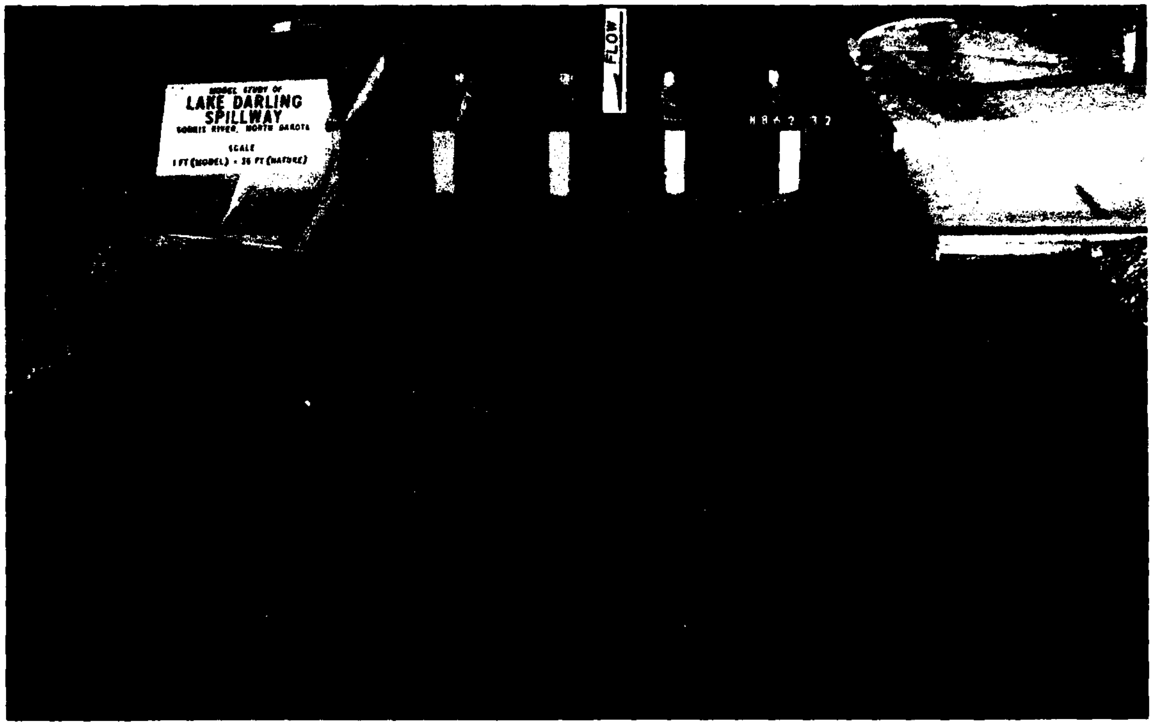
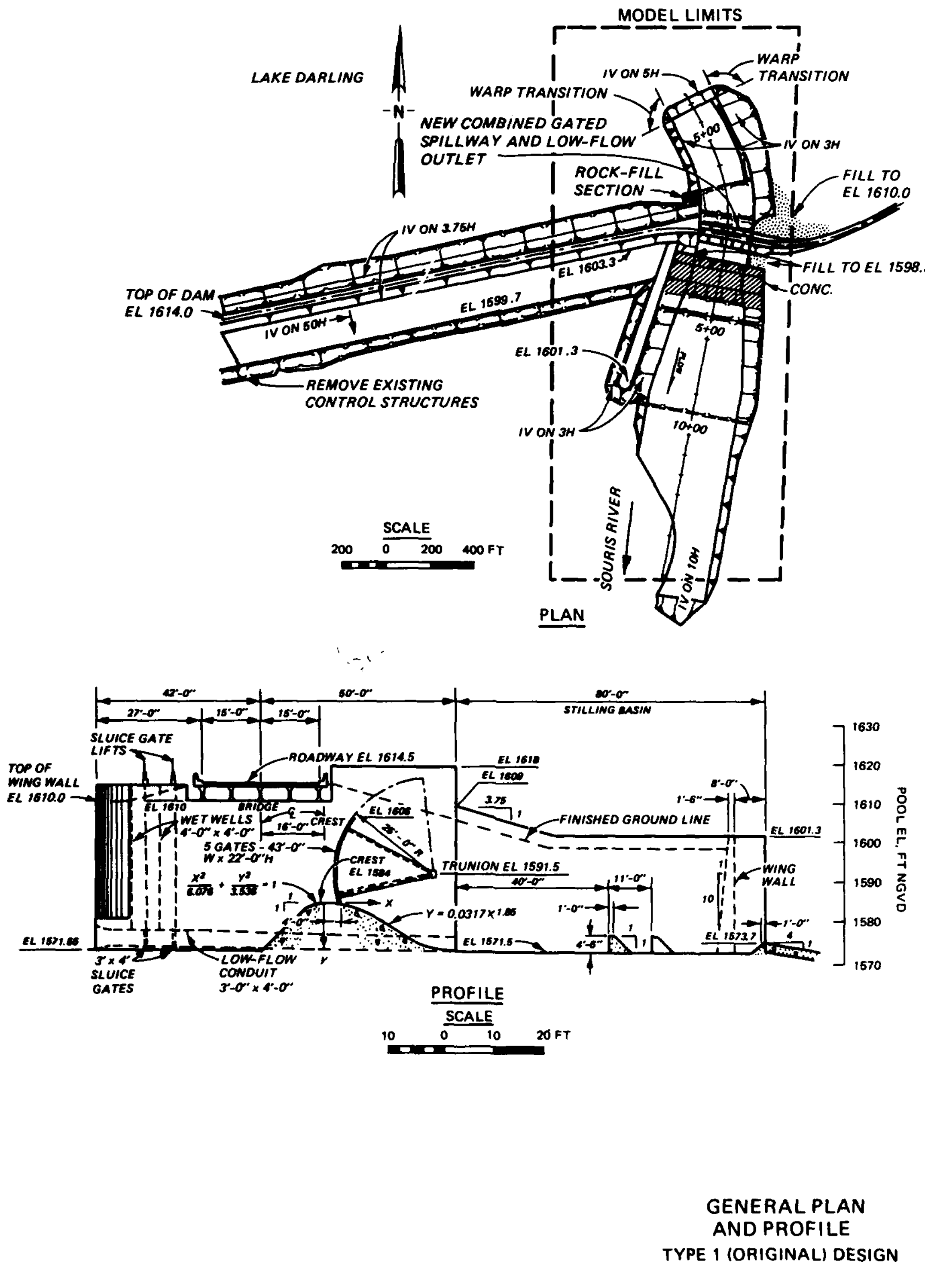
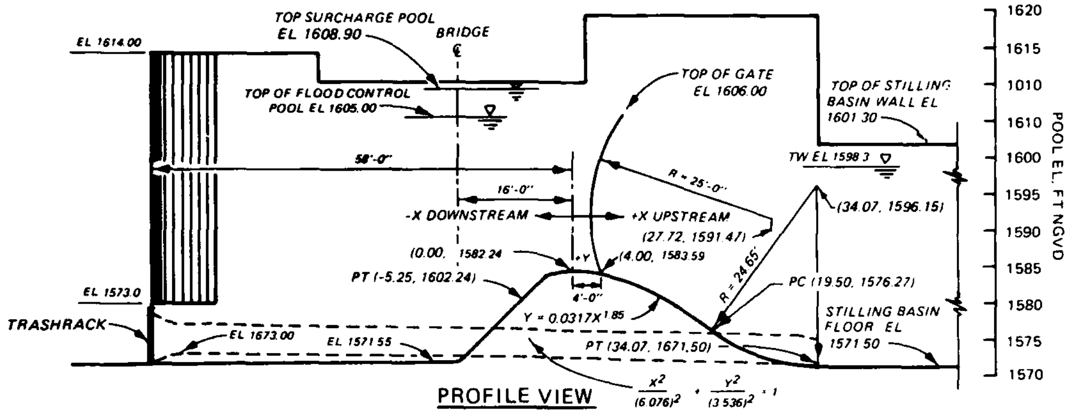



Photo 5. Type 1 stilling basin; type 1 (original) left bank approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates fully open


Photo 6. Type 2 stilling basin; type 1 (original) left bank approach; discharge 99,800 cfs; pool el 1,609.0; tailwater el 1,598.3; all gates fully open

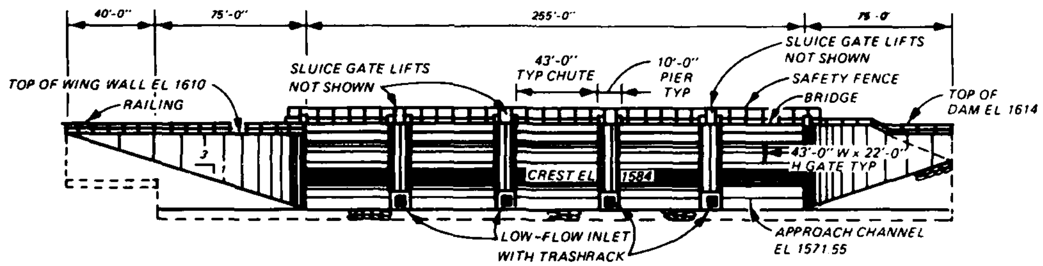



GENERAL PLAN
AND PROFILE
TYPE 1 (ORIGINAL) DESIGN




## PROFILE VIEW

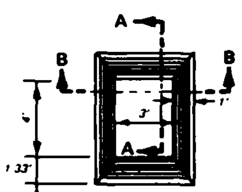
SCALE 1/4" = 5'-0"




## UPSTREAM ELEVATION

SCALE

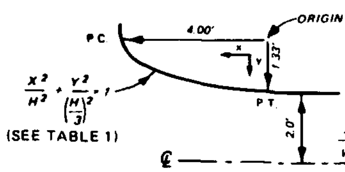
20 0 20 40 FT




FLUSH SLUICE INTAKE

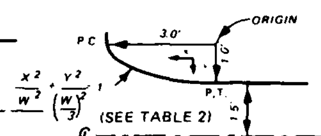
## DETAIL "A"

SCALE 5/16" = 1'-0"




## SECTION A-A

SCALE 1/4" = 1'-0"




## SECTION B-B

SCALE 1/4" = 1'-0"

**TABLE 1**

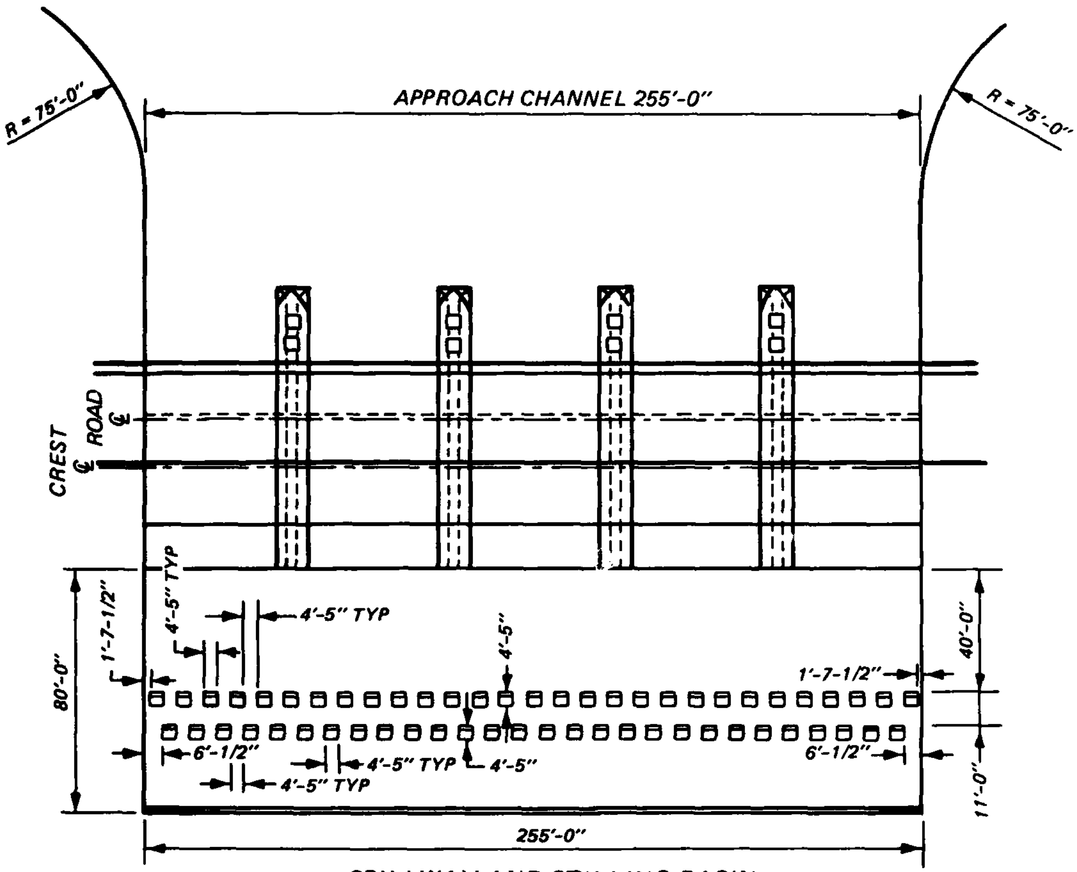
| $x^2/(4)^2 + y^2/(4/3)^2 = 1$ | |
|---|---|
| X | Y |
| 0.00 | 1.33 |
| 0.25 | 1.33 |
| 0.50 | 1.32 |
| 0.75 | 1.31 |
| 1.00 | 1.29 |
| 1.25 | 1.27 |
| 1.50 | 1.24 |
| 1.75 | 1.20 |
| 2.00 | 1.15 |
| 2.25 | 1.10 |
| 2.50 | 1.04 |
| 2.75 | 0.97 |
| 3.00 | 0.88 |
| 3.25 | 0.77 |
| 3.50 | 0.64 |
| 3.75 | 0.46 |
| 4.00 | 0.00 |

**TABLE 2**

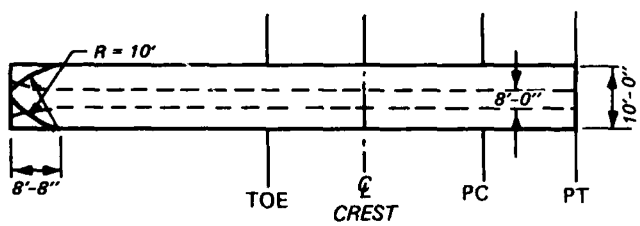
| $x^2/(3)^2 + y^2/(3/3)^2 = 1$ | |
|---|---|
| X | Y |
| 0.00 | 1.00 |
| 0.25 | 1.00 |
| 0.50 | 0.99 |
| 0.75 | 0.97 |
| 1.00 | 0.94 |
| 1.25 | 0.91 |
| 1.50 | 0.87 |
| 1.75 | 0.81 |
| 2.00 | 0.74 |
| 2.25 | 0.66 |
| 2.50 | 0.55 |
| 2.75 | 0.40 |
| 3.00 | 0.00 |

ELEVATION AND PROFILE VIEWS
LOW-FLOW SLUICES

PLATE 2

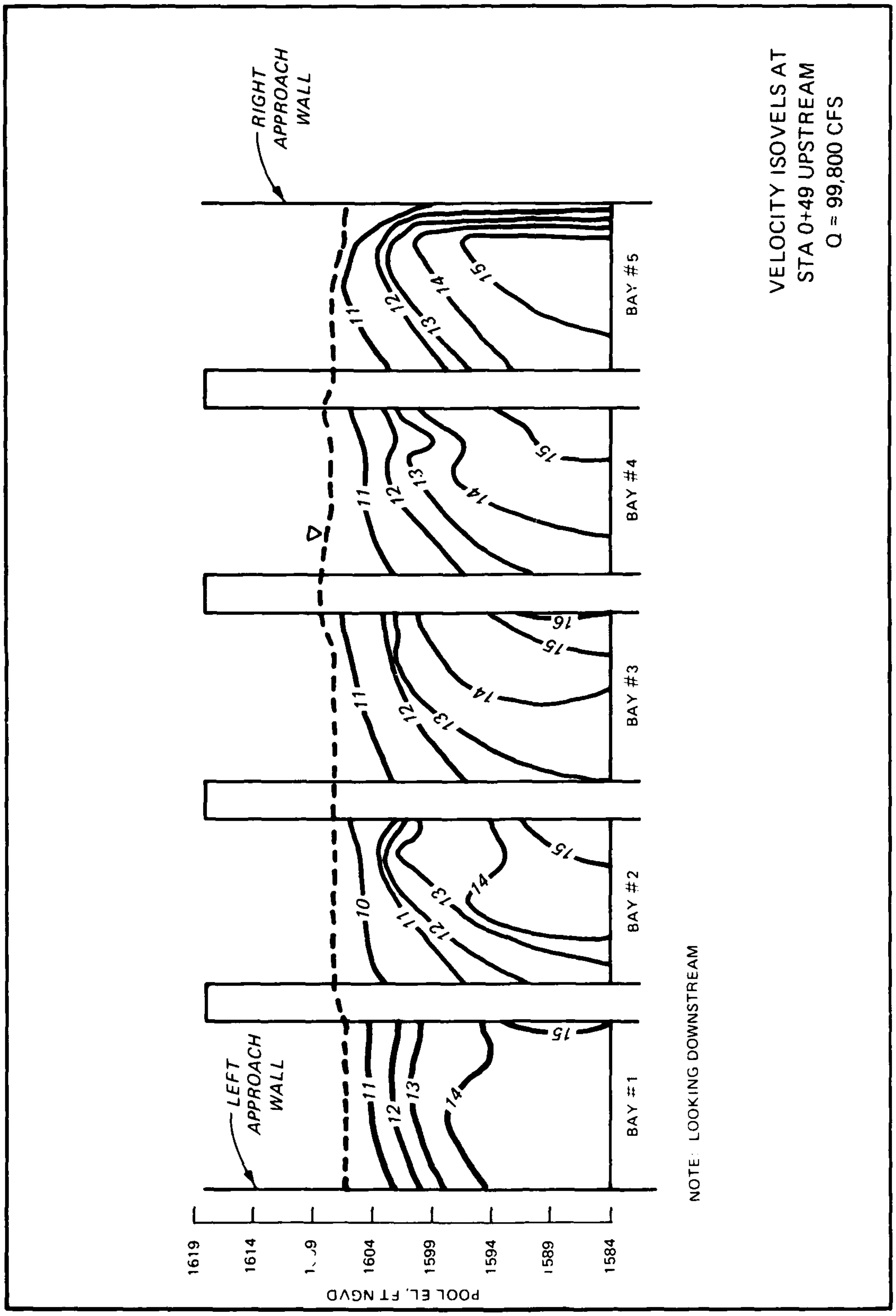



SPILLWAY AND STILLING BASIN
PLAN
SCALE: 3/8" = 20'-0"



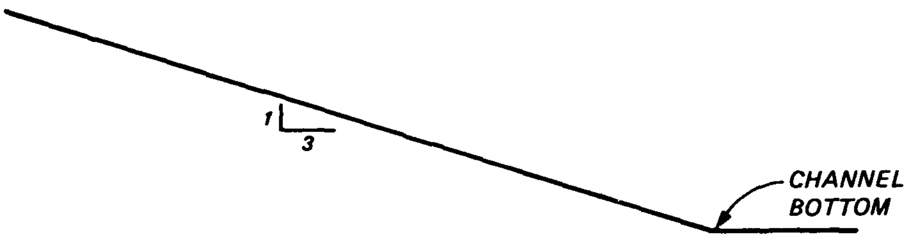

(GATES NOT SHOWN)
PIER PLAN VIEW
SCALE: 3/8" = 10'-0"

ORIGINAL STILLING BASIN AND
PIER DETAILS
ORIGINAL DESIGN

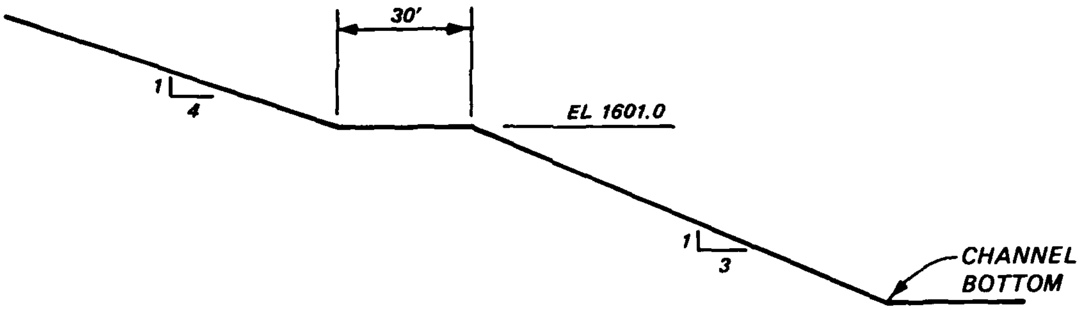


VELOCITY ISOVELS AT
STA 0+49 UPSTREAM
Q = 99,800 CFS
RIGHT APPROACH WALL
LEFT APPROACH WALL
BAY #1
BAY #2
BAY #3
BAY #4
BAY #5
NOTE: LOOKING DOWNSTREAM
POOL EL, FT NGVD
1619
1614
1604
1599
1594
1589
1584


PLATE 4

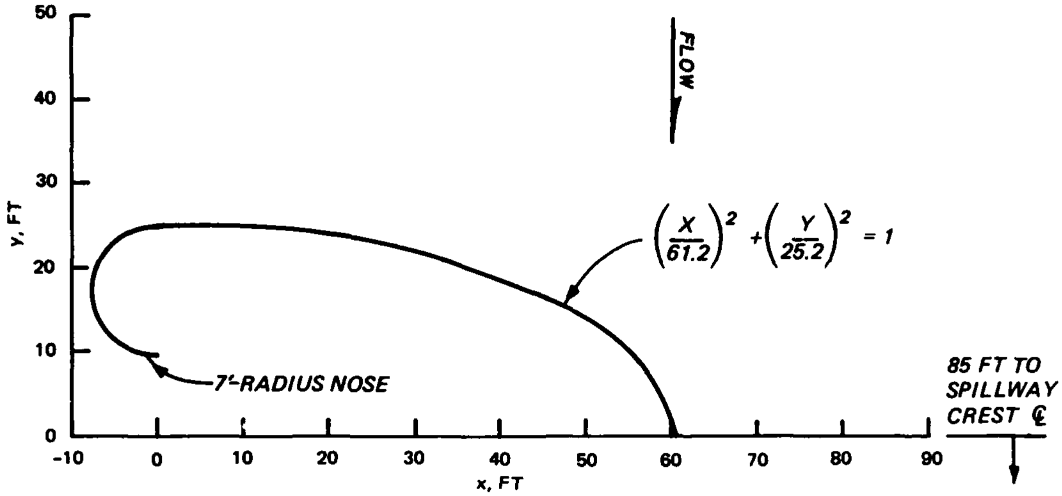



TYPICAL CROSS SECTION
TYPE 1 (ORIGINAL) DESIGN APPROACH

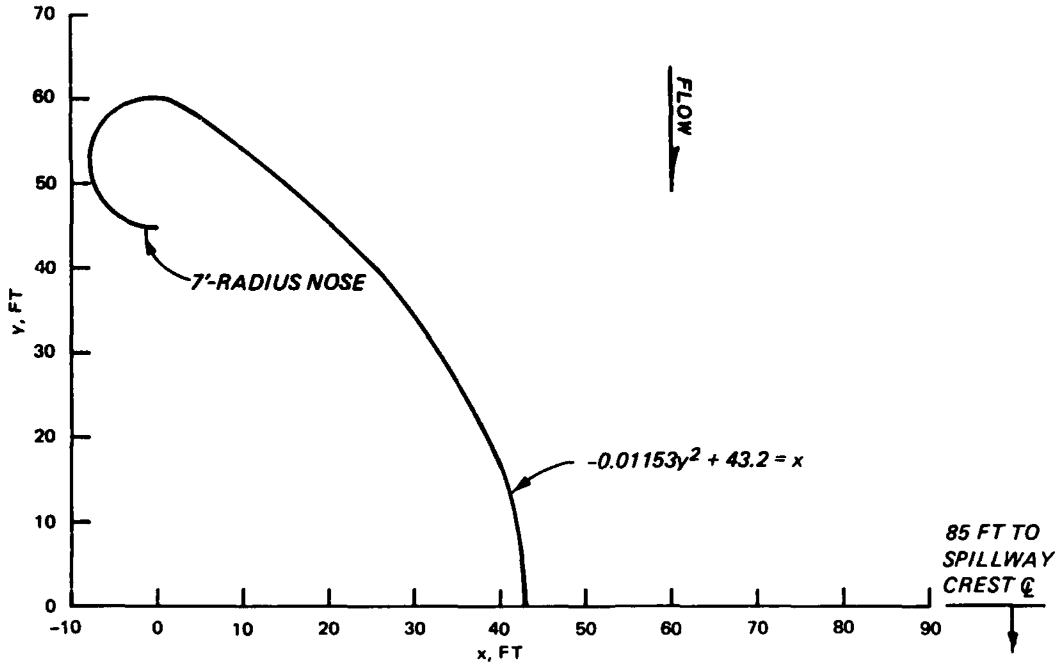



TYPICAL CROSS SECTION
TYPE 2 DESIGN APPROACH

TYPICAL CROSS SECTIONS
TYPE 1 (ORIGINAL)
AND TYPE 2
LEFT BANK APPROACH

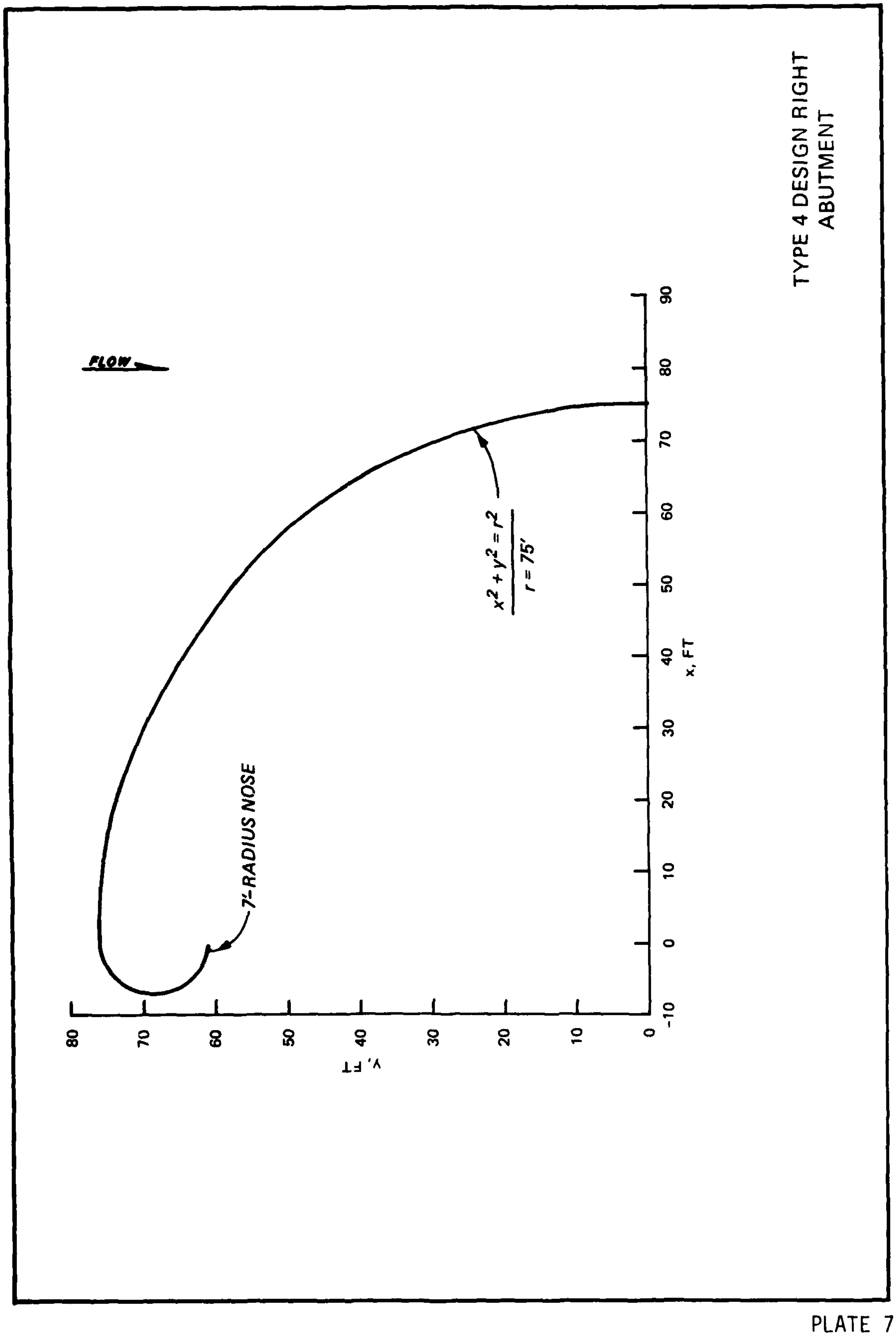



TYPE 2 (ELLIPTICAL) WALL

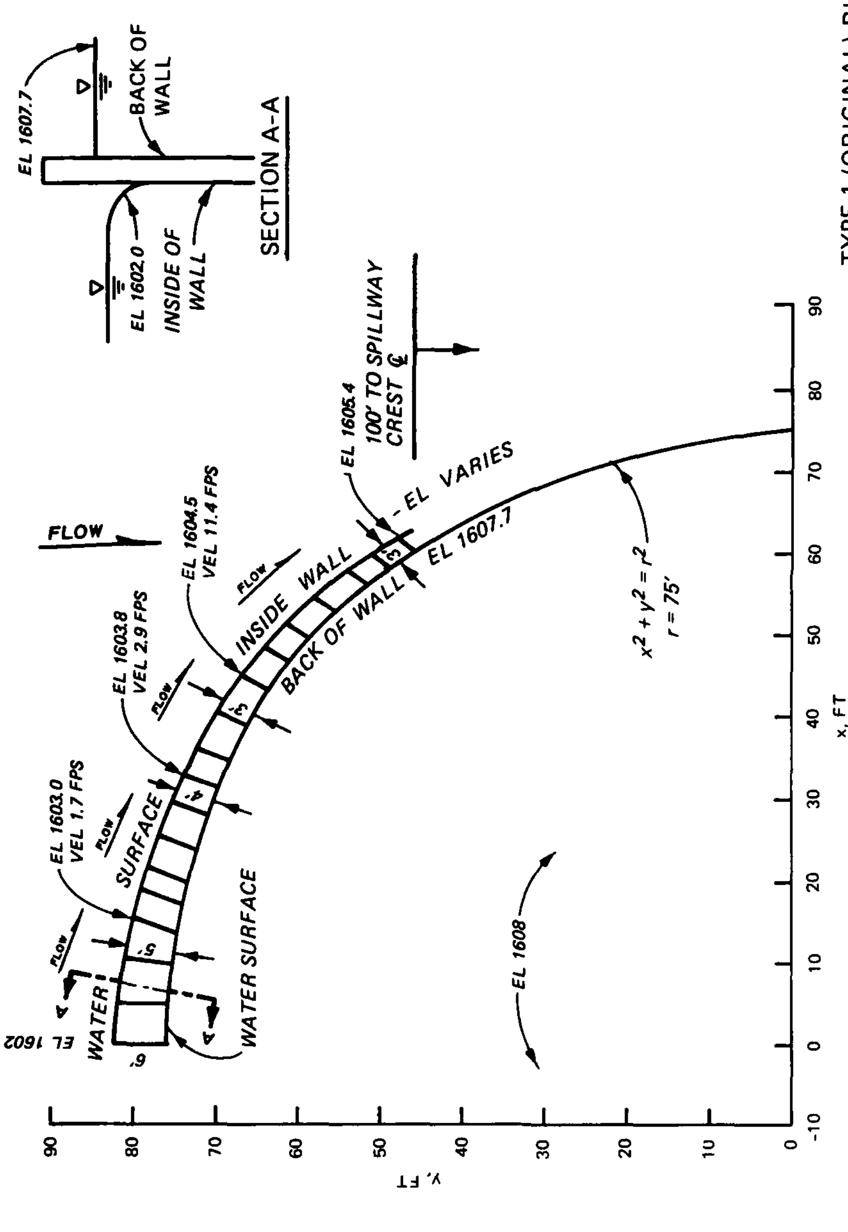



TYPE 3 (PARABOLIC) WALL

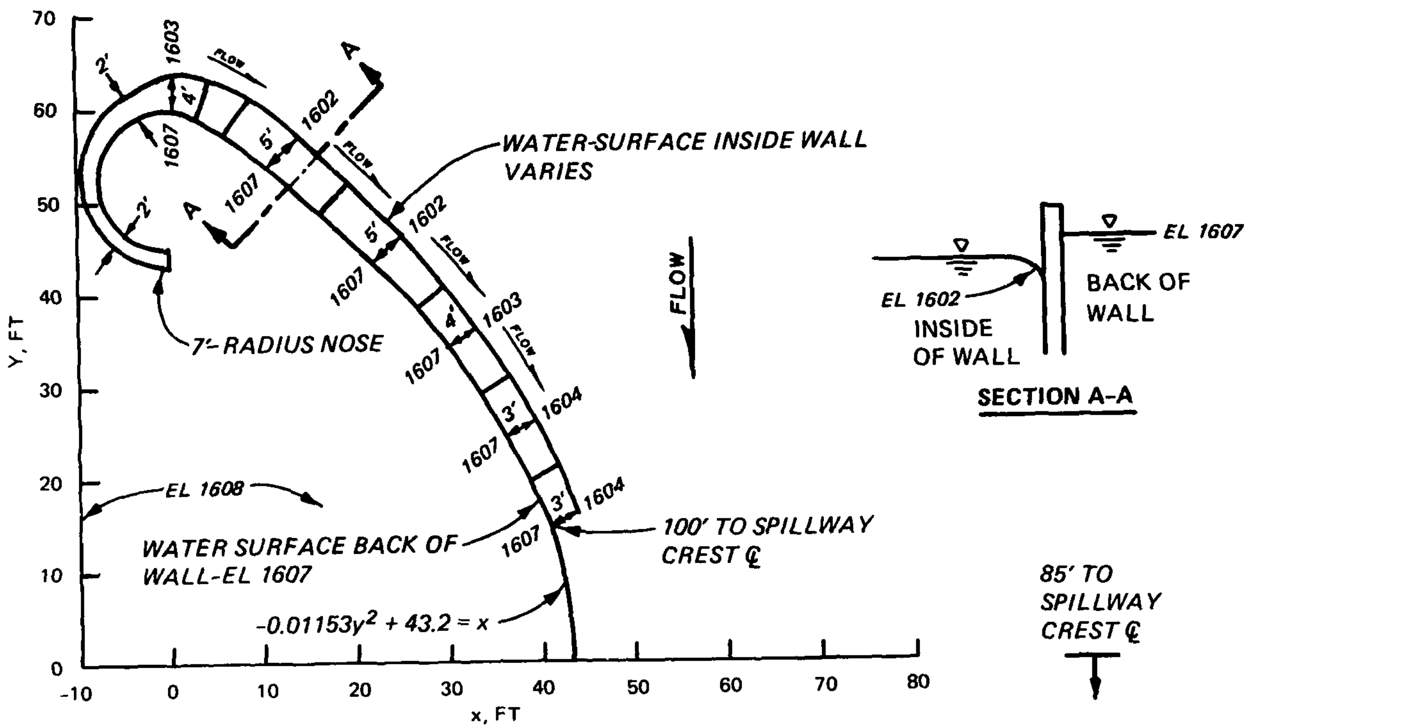
TYPES 2 AND 3 WALLS




TYPE 4 DESIGN RIGHT ABUTMENT




TYPE 1 (ORIGINAL) RIGHT WALL
WATER-SURFACE DIFFERENTIALS
TYPE 1 (ORIGINAL) LEFT
BANK APPROACH
DISCHARGE - 99,800 CFS
POOL EL 1609 NGVD
TW EL 1598.3 NGVD




TYPE 3 (PARABOLIC) WALL

TYPE 3 DESIGN (PARABOLIC) WALL
WATER-SURFACE DIFFERENTIALS
TYPE 1 (ORIGINAL) LEFT BANK APPROACH
DISCHARGE - 99,800 CFS
POOL EL 1609 NGVD
TW EL 1598.3 NGVD

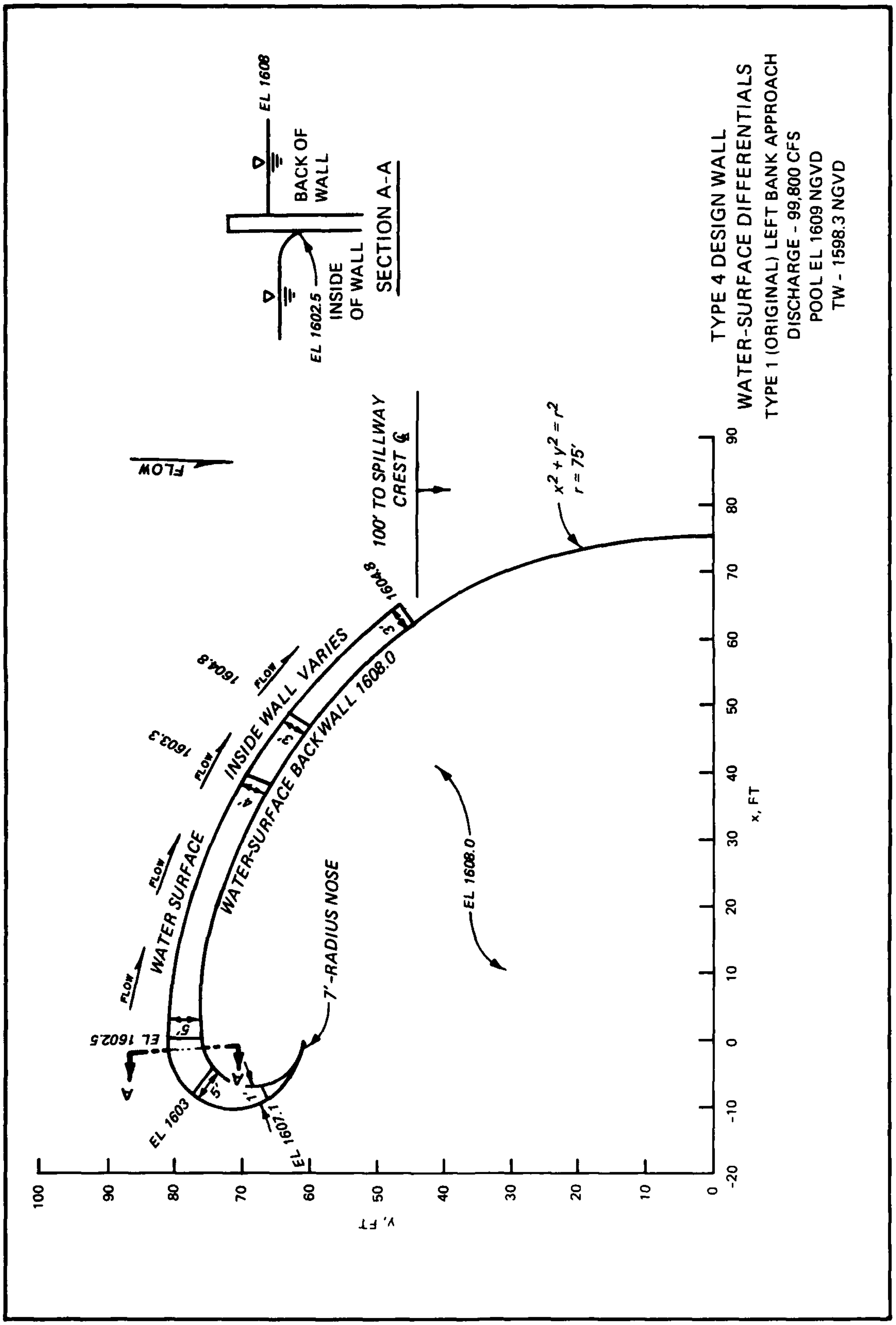
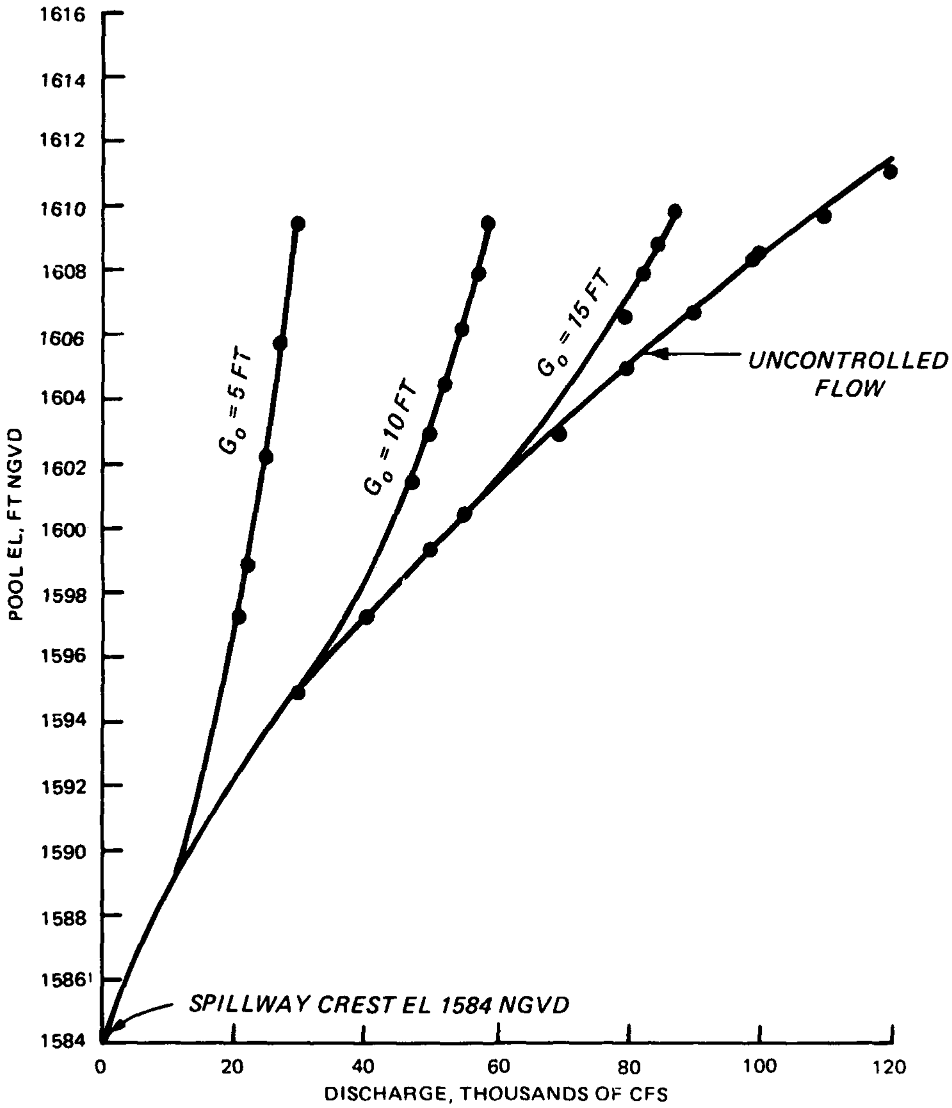
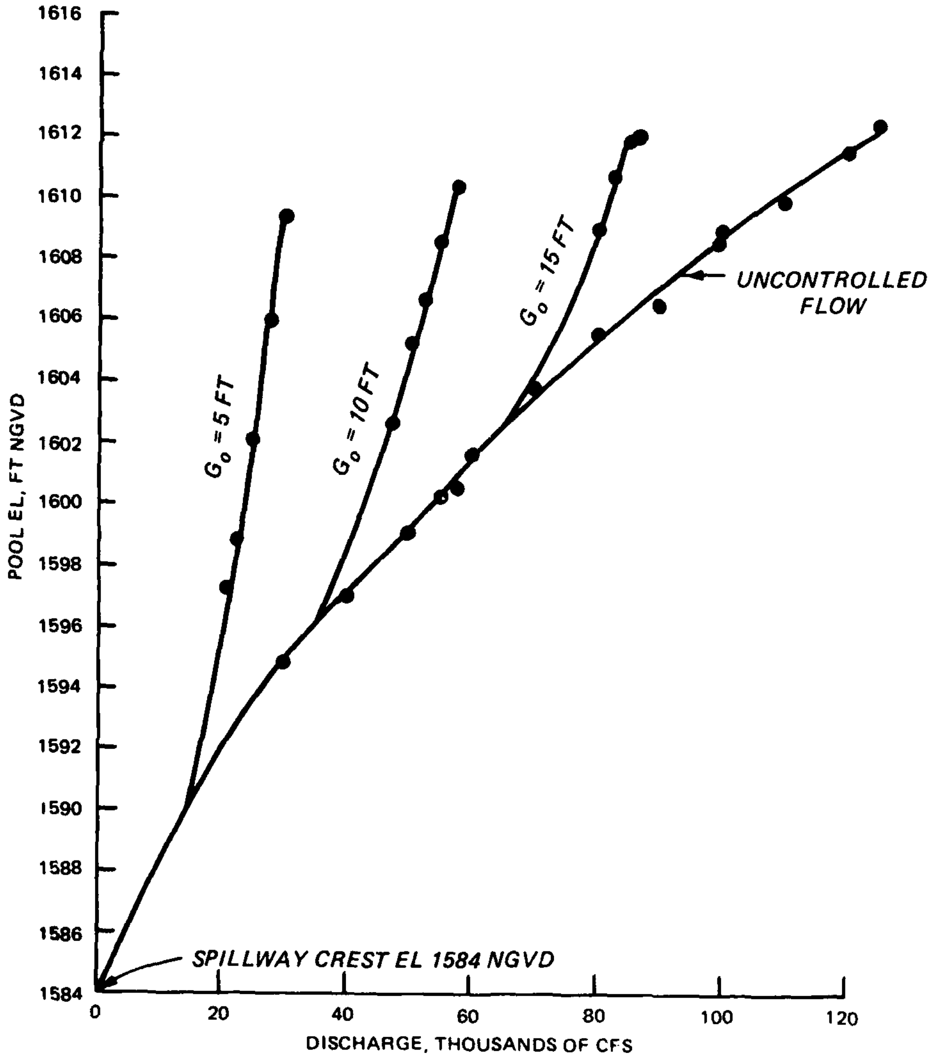

TYPE 4 DESIGN WALL
WATER-SURFACE DIFFERENTIALS
TYPE 1 (ORIGINAL) LEFT BANK APPROACH
DISCHARGE - 99,800 CFS
POOL EL 1609 NGVD
TW - 1598.3 NGVD
SECTION A-A
BACK OF WALL
INSIDE OF WALL
EL 1608
EL 1602.5
FLOW
100' TO SPILLWAY CREST ℄
$x^2 + y^2 = r^2$
$r = 75'$
INSIDE WALL VARIES
WATER-SURFACE BACKWALL 1608.0
WATER SURFACE
1604.8
1603.3
EL 1608.0
7'-RADIUS NOSE
EL 1602.5
EL 1603
EL 1607.1
A
x, FT
y, FT

PLATE 10



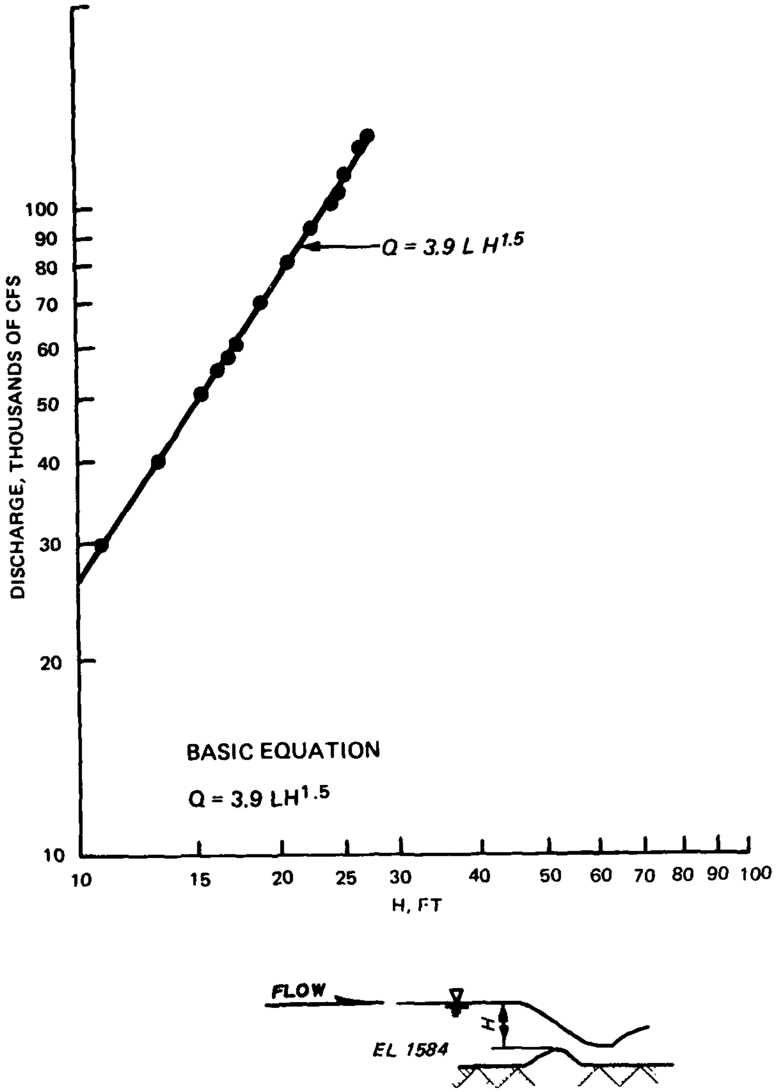

RATING CURVE
TYPE 1 (ORIGINAL) LEFT
BANK APPROACH




NOTE: $G_o$ = GATE OPENING

RATING CURVE

TYPE 2 LEFT BANK APPROACH

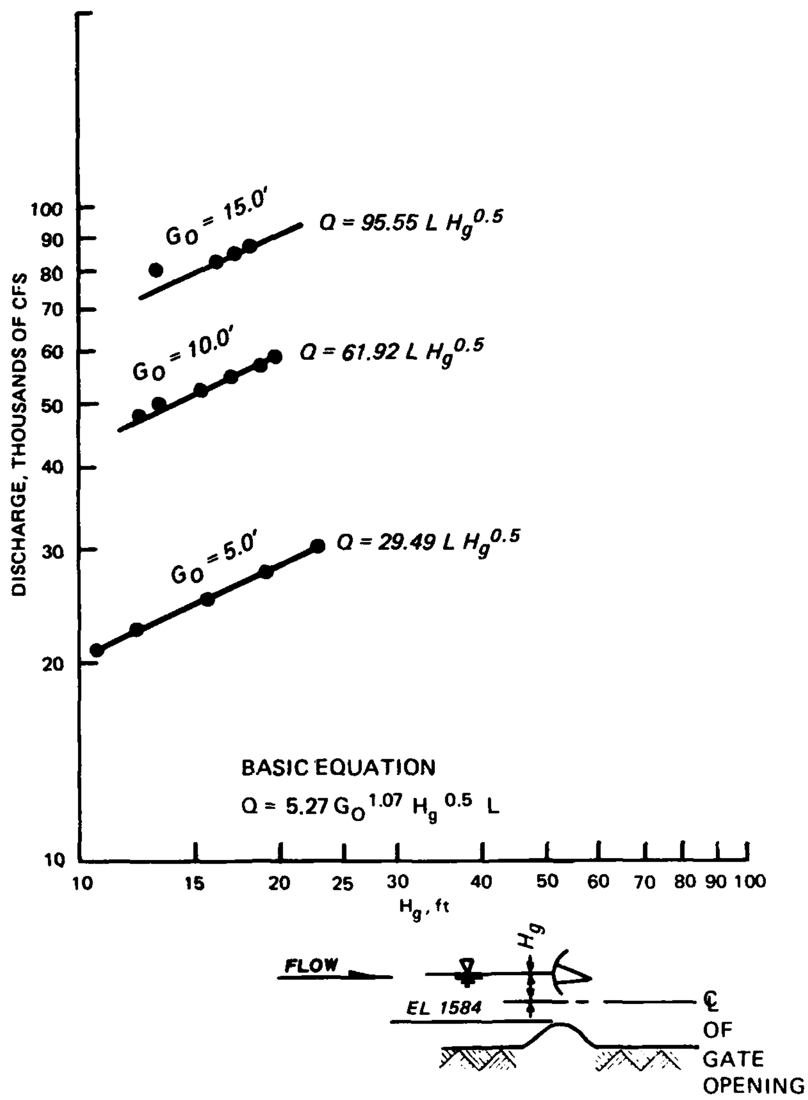



NOTE: $Q$ = DISCHARGE, CFS

$L$ = NET LENGTH OF SPILLWAY, FT

$H$ = TOTAL HEAD ON THE CREST, FT

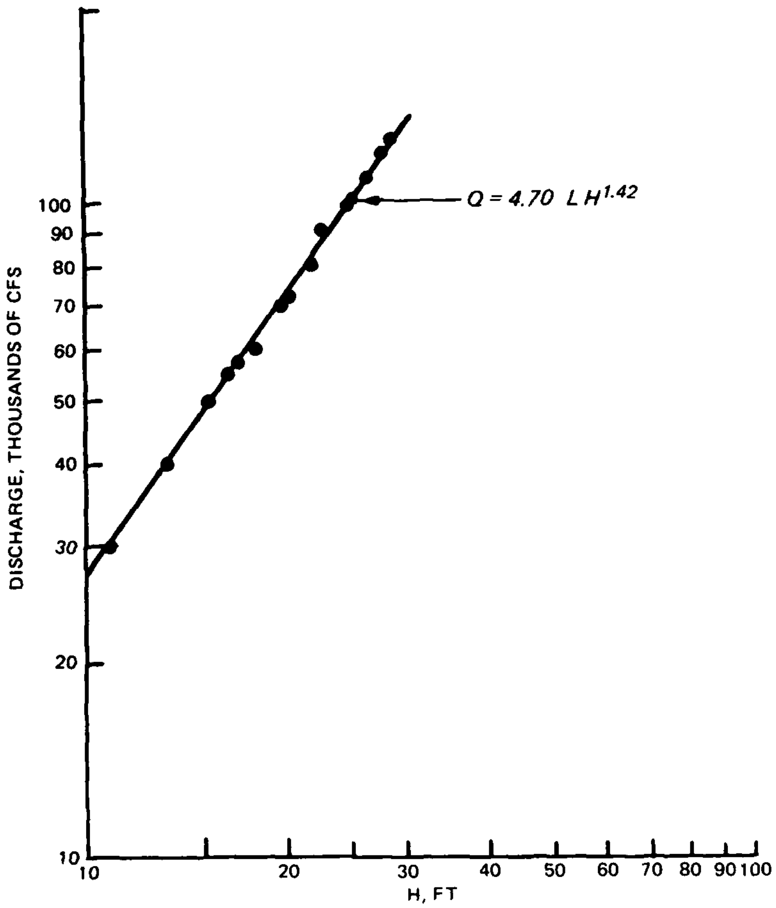
DISCHARGE VERSUS TOTAL HEAD
FOR
UNCONTROLLED FLOW
ORIGINAL LEFT BANK APPROACH

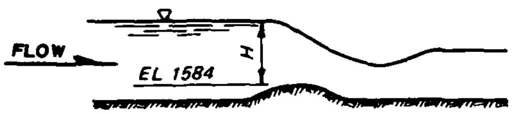



NOTE: $H_g$ = HEAD ON THE GATE, FT

DISCHARGE VERSUS HEAD ON GATE FOR CONTROLLED FLOW

ORIGINAL LEFT BANK APPROACH

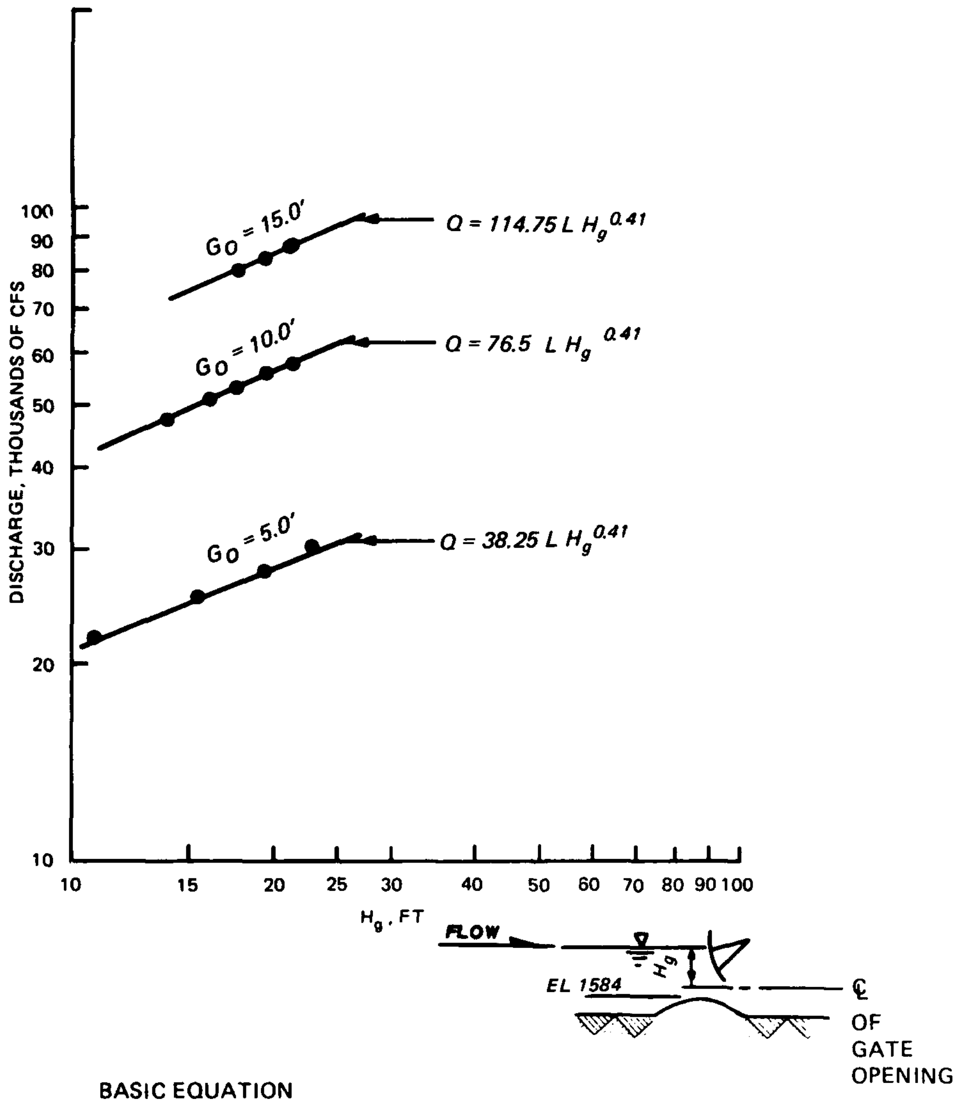



BASIC EQUATION

$Q = 4.70\,LH^{1.42}$

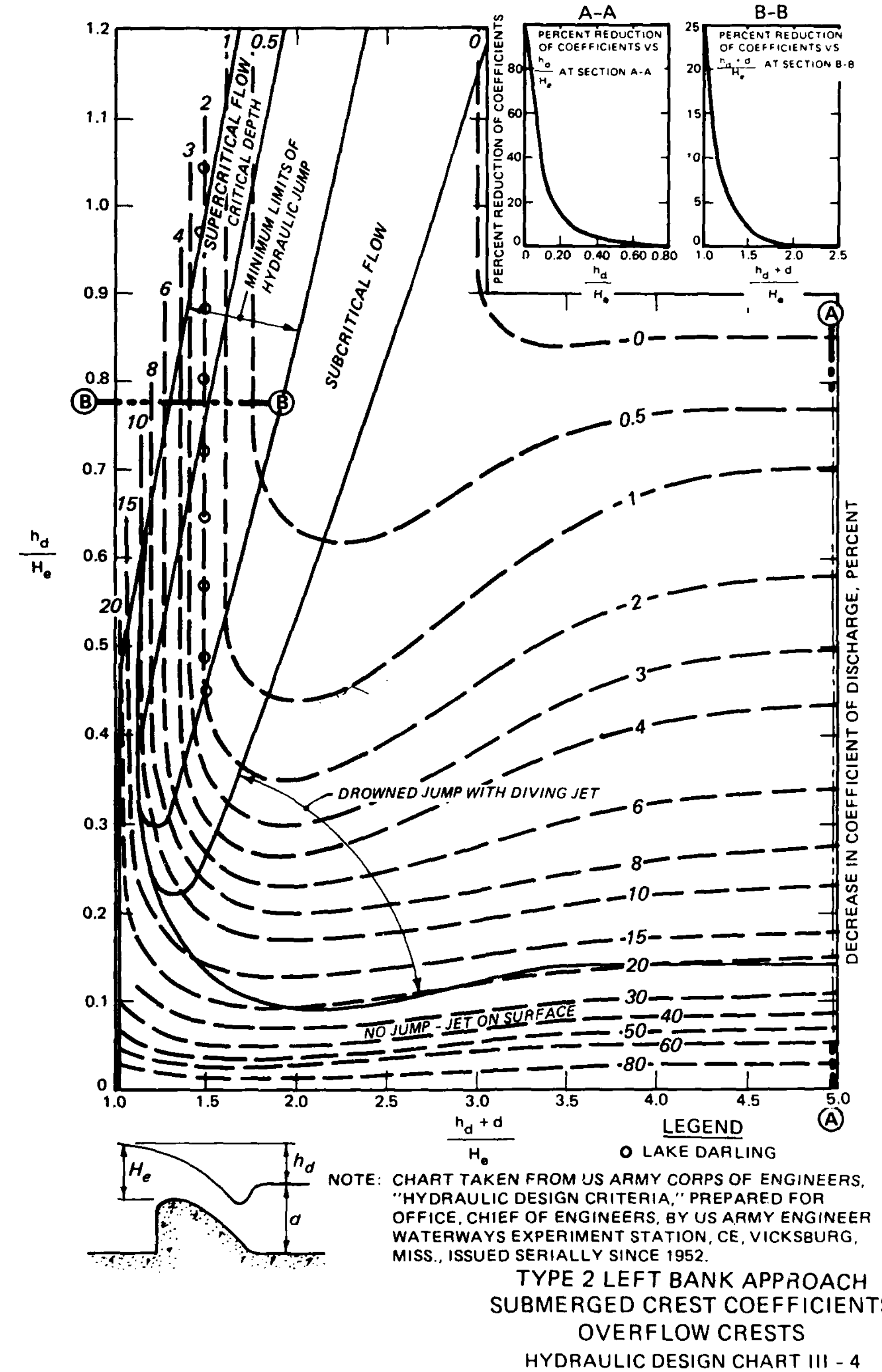
DISCHARGE VERSUS TOTAL HEAD
FOR
UNCONTROLLED FLOW
TYPE 2 LEFT BANK APPROACH




BASIC EQUATION

$Q = 7.65\ G_O^{1.0}\ H_g^{0.41}\ L$

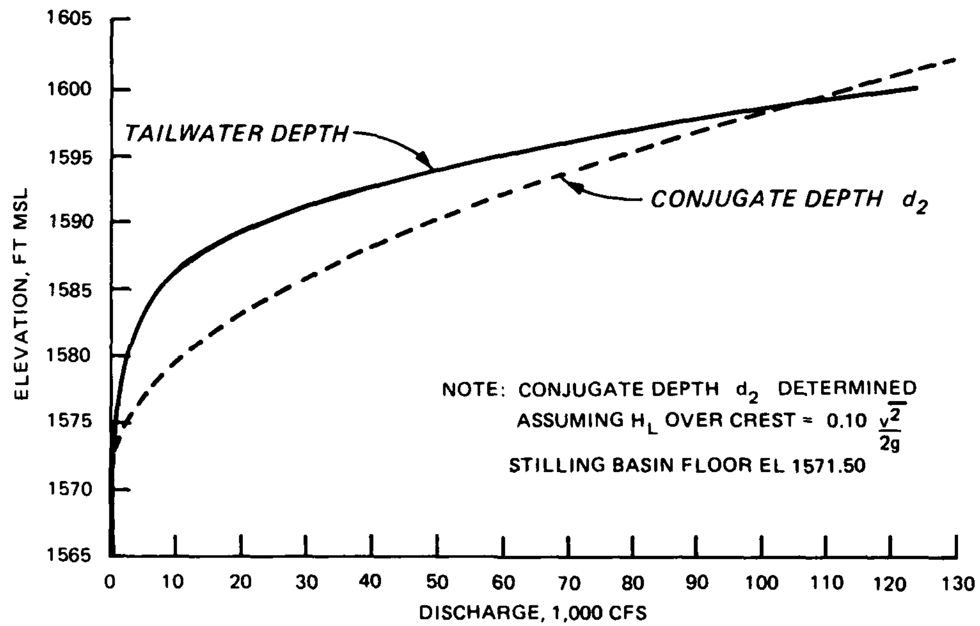
DISCHARGE VERSUS HEAD ON GATE
FOR
CONTROLLED FLOW
TYPE 2 LEFT BANK APPROACH




NOTE: CHART TAKEN FROM US ARMY CORPS OF ENGINEERS, "HYDRAULIC DESIGN CRITERIA," PREPARED FOR OFFICE, CHIEF OF ENGINEERS, BY US ARMY ENGINEER WATERWAYS EXPERIMENT STATION, CE, VICKSBURG, MISS., ISSUED SERIALLY SINCE 1952.

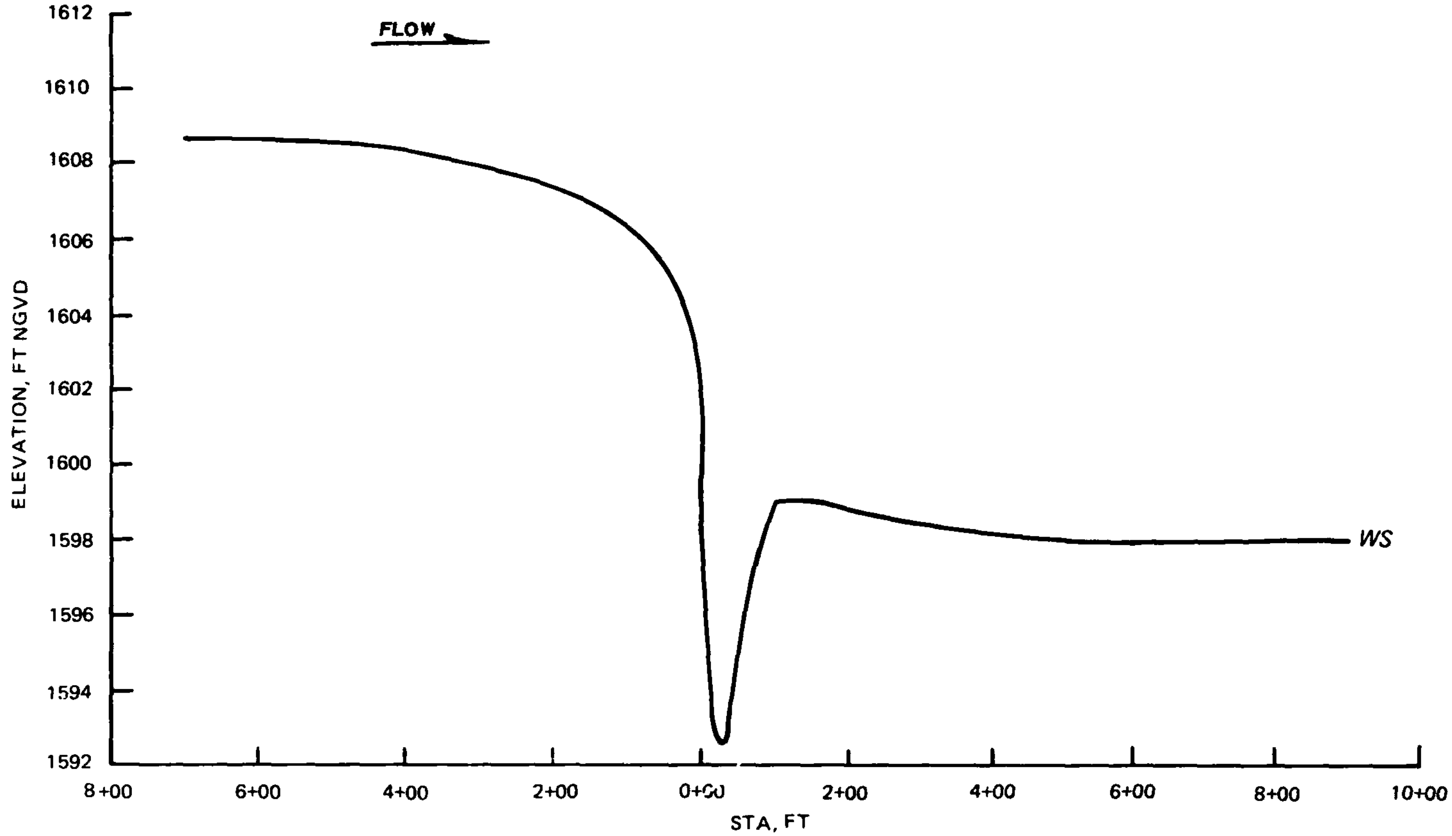
TYPE 2 LEFT BANK APPROACH
SUBMERGED CREST COEFFICIENTS
OVERFLOW CRESTS

HYDRAULIC DESIGN CHART III - 4



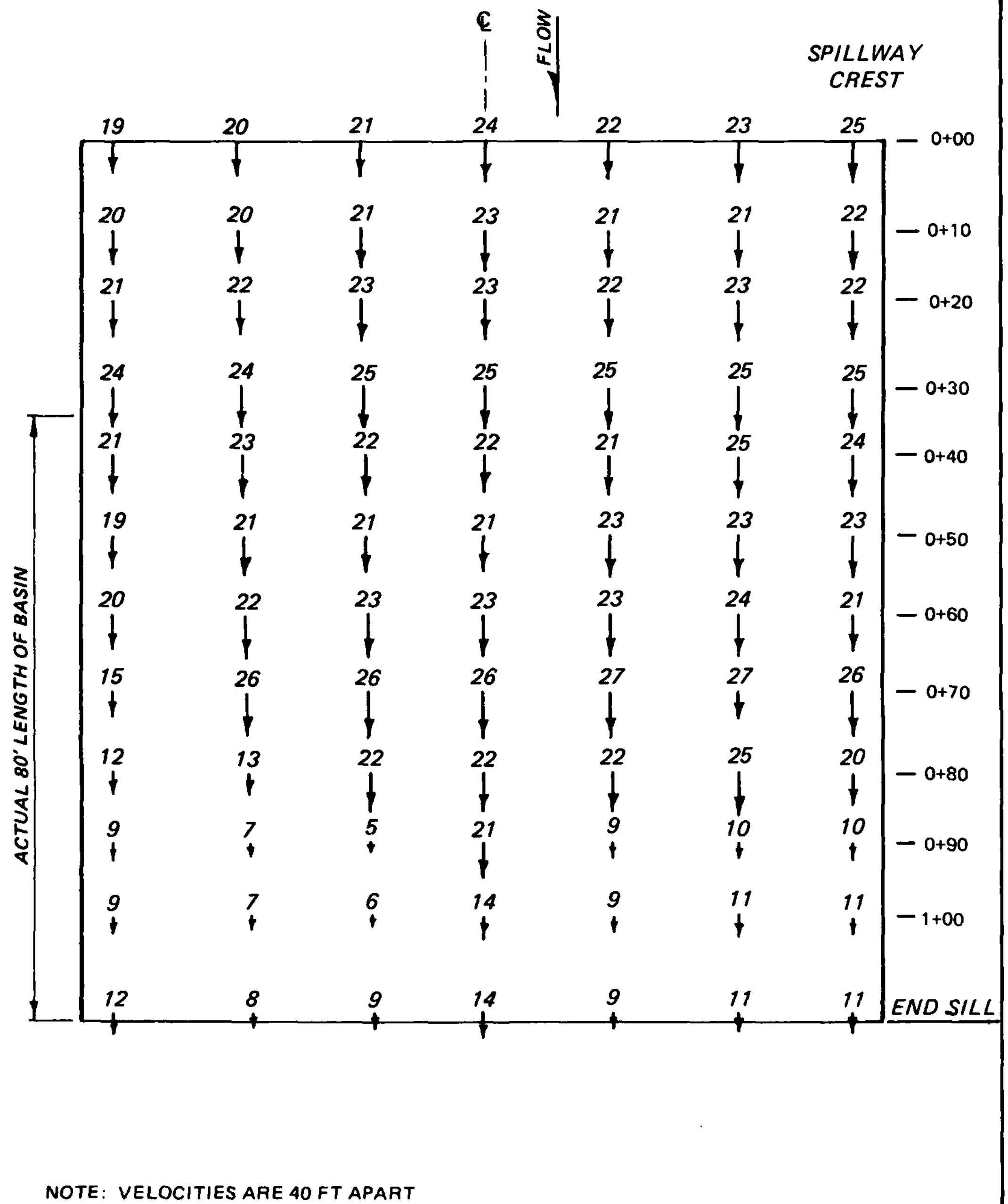

LAKE DARLING TAILWATER RATING CURVE
PROPOSED CONDITIONS

EXPECTED TAILWATER CURVE

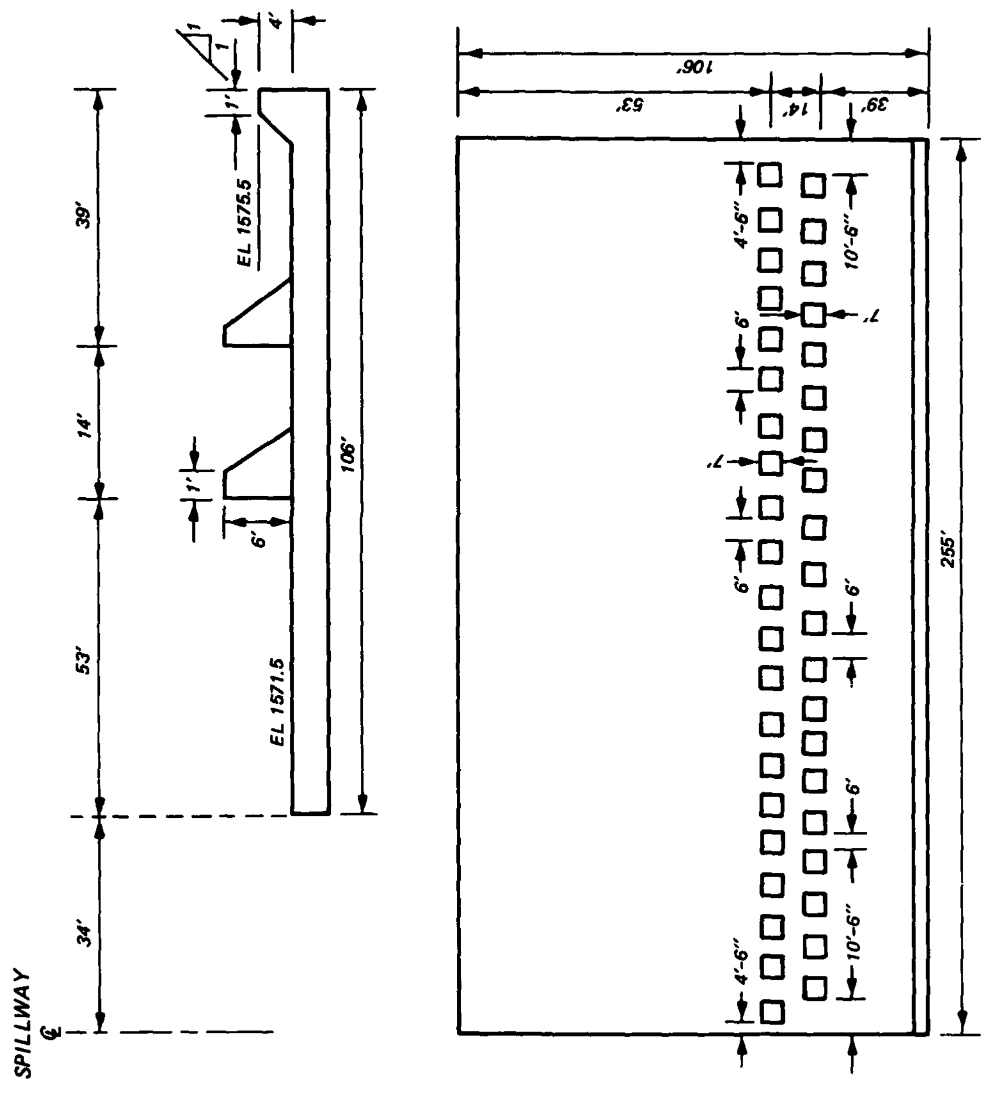



TYPE 1 (ORIGINAL) LEFT BANK APPROACH

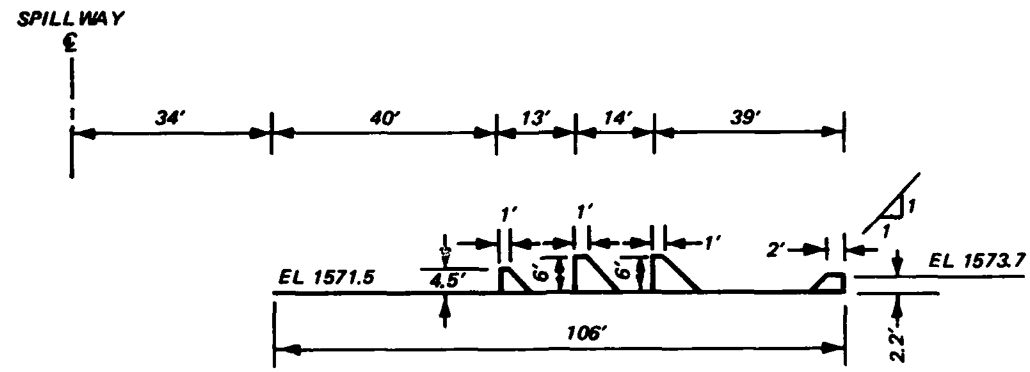
WATER-SURFACE PROFILE
TYPE I STILLING BASIN
Q = 99,800 CFS
POOL 1609 NGVD
TW 1598.3 NGVD



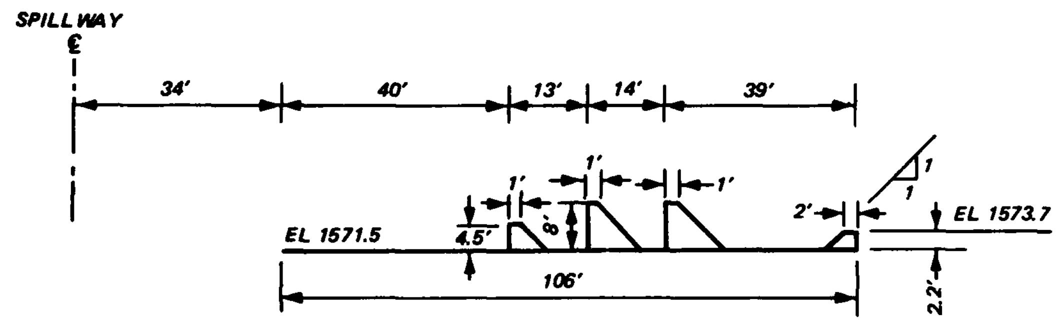

STILLING BASIN BOTTOM VELOCITIES
TYPE 1 (ORIGINAL) LEFT BANK APPROACH DESIGN
AND STILLING BASIN
Q = 99,800 CFS
SCALE 1:36



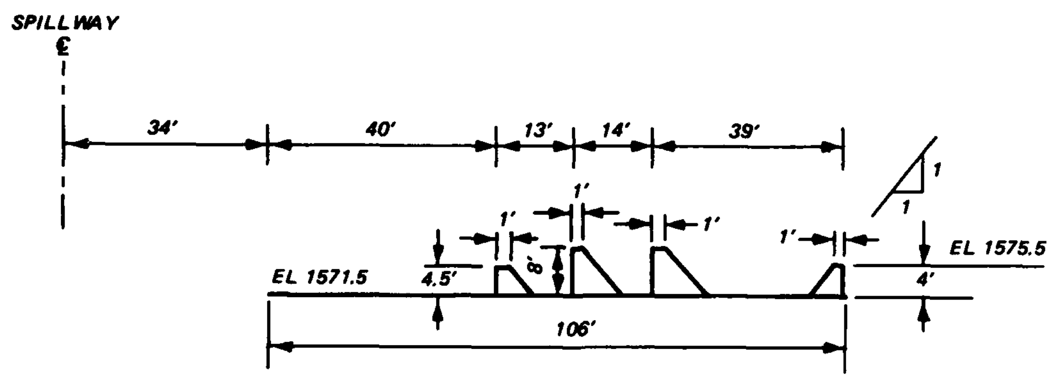

TYPE 2 DESIGN STILLING BASIN




TYPE 3 STILLING BASIN
PROFILE VIEW

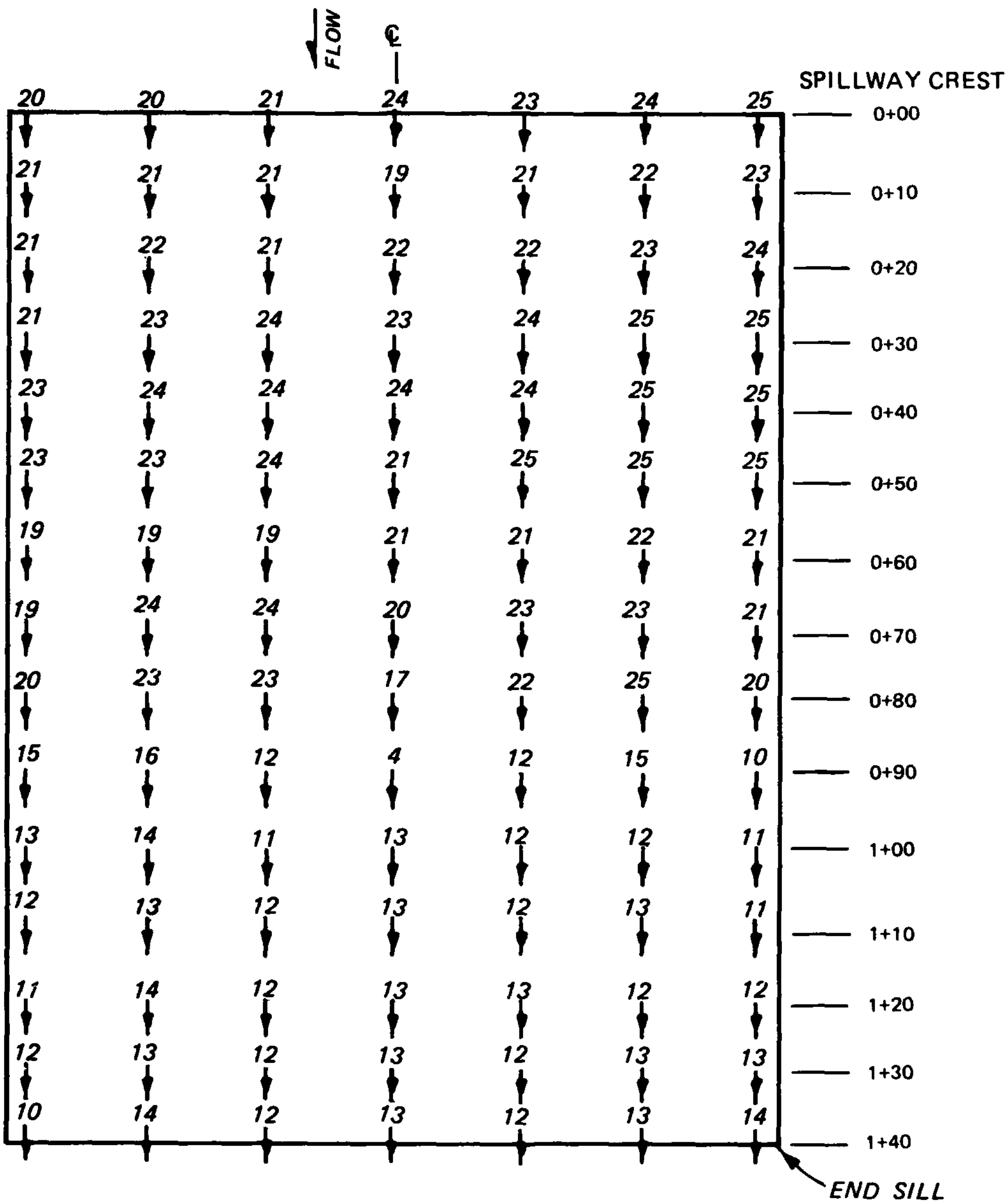



TYPE 4 STILLING BASIN
PROFILE VIEW



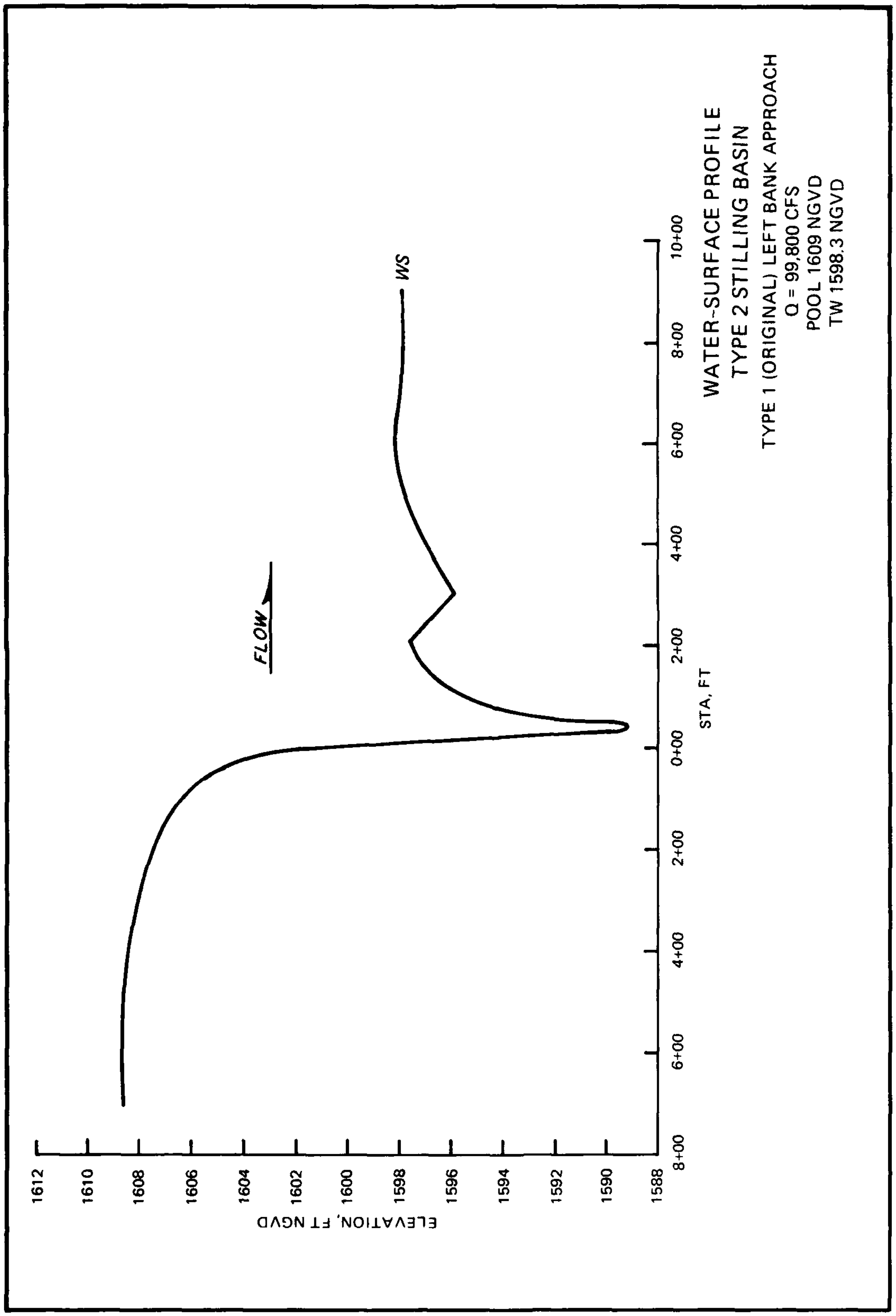

TYPE 5 STILLING BASIN
PROFILE VIEW
SCALE 1/32" = 1'-0"

TYPES 3, 4, AND 5
STILLING BASINS



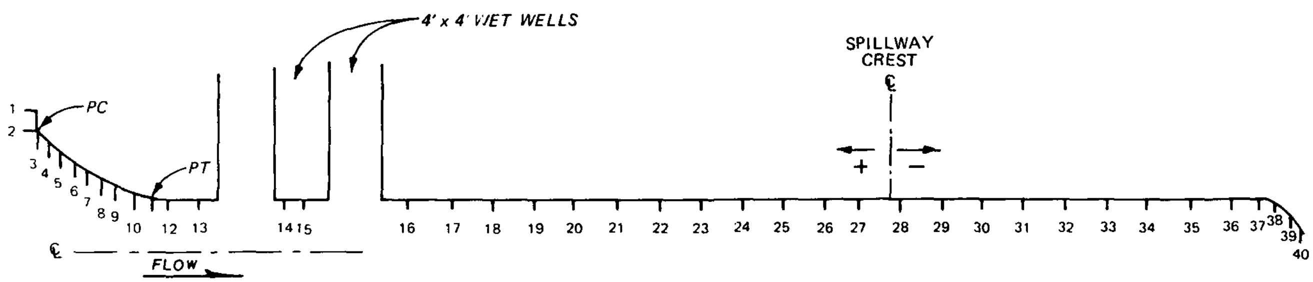

NOTE: VELOCITIES ARE 40 FT APART

STILLING BASIN BOTTOM VELOCITIES
TYPE 1 (ORIGINAL) LEFT BANK APPROACH
TYPE 2 DESIGN
STILLING BASIN
Q = 99,800 CFS

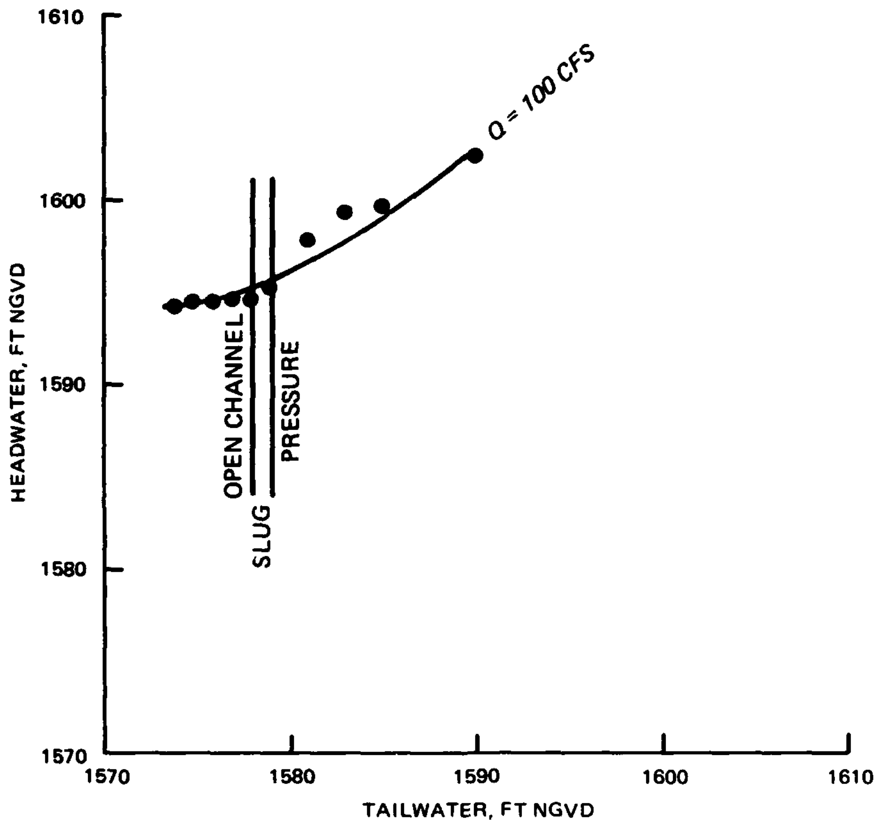

WATER-SURFACE PROFILE
TYPE 2 STILLING BASIN
TYPE 1 (ORIGINAL) LEFT BANK APPROACH
Q = 99,800 CFS
POOL 1609 NGVD
TW 1598.3 NGVD
ELEVATION, FT NGVD
STA, FT
FLOW
WS
1612
1610
1608
1606
1604
1602
1600
1598
1596
1594
1592
1590
1588
8+00
6+00
4+00
2+00
0+00
2+00
4+00
6+00
8+00
10+00

LOCATION OF PIEZOMETER TUBES
SCALE 1:12

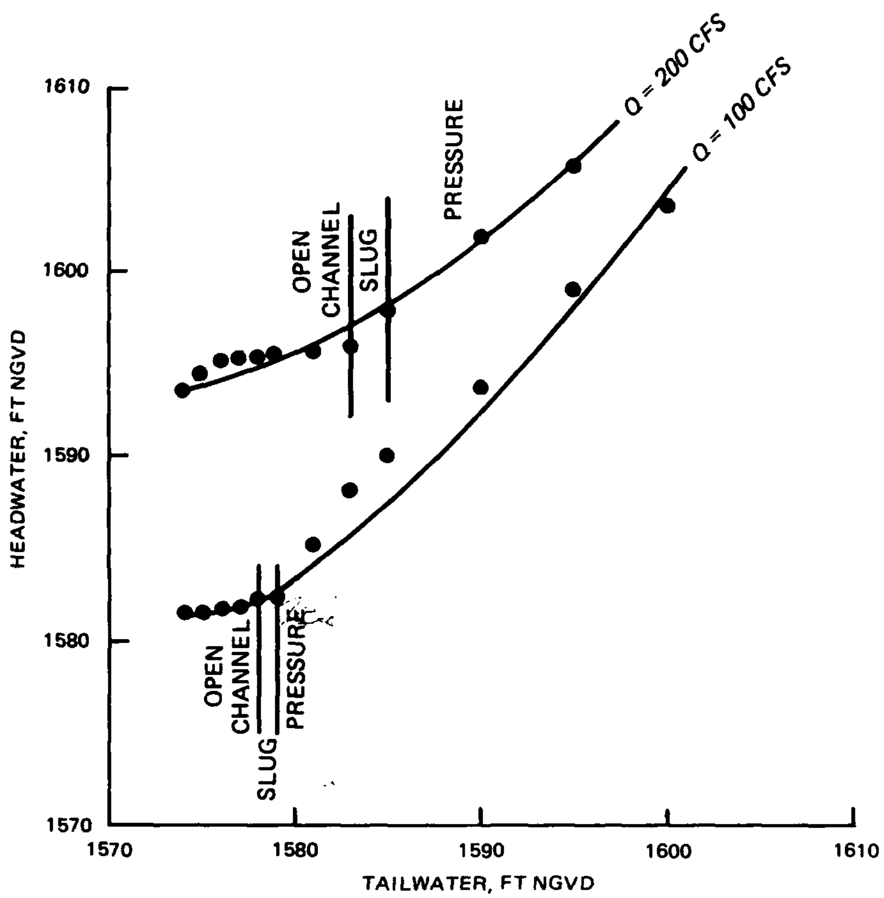
| PIEZOMETER NO. | STA | EL | PIEZOMETER NO. | STA | EL |
|---|---|---|---|---|---|
| 1 | 0+58.0 | 1578.8 | 21 | 0+20 | 1577.0 |
| 2 | 0+58.0 | 1578.3 | 22 | 0+17 | 1577.0 |
| 3 | 0+57.7 | 1577.9 | 23 | 0+14 | 1577.0 |
| 4 | 0+57.5 | 1577.7 | 24 | 0+11 | 1577.0 |
| 5 | 0+57.0 | 1577.5 | 25 | 0+08 | 1577.0 |
| 6 | 0+56.5 | 1577.3 | 26 | 0+05 | 1577.0 |
| 7 | 0+56.0 | 1577.2 | 27 | 0+02 | 1577.0 |
| 8 | 0+55.5 | 1577.1 | 28 | -0+01 | 1577.0 |
| 9 | 0+55.0 | 1577.0 | 29 | -0+04 | 1577.0 |
| 10 | 0+54.5 | 1577.0 | 30 | -0+07 | 1577.0 |
| 11 | 0+54.0 | 1577.0 | 31 | -0+10 | 1577.0 |
| 12 | 0+53.0 | 1577.0 | 32 | -0+13 | 1577.0 |
| 13 | 0+50.0 | 1577.0 | 33 | -0+16 | 1577.0 |
| 14 | 0+44.0 | 1577.0 | 34 | -0+19 | 1577.0 |
| 15 | 0+43.0 | 1577.0 | 35 | -0+22 | 1577.0 |
| 16 | 0+35.0 | 1577.0 | 36 | -0+25 | 1577.0 |
| 17 | 0+32.0 | 1577.0 | 37 | -0+28 | 1577.0 |
| 18 | 0+29.0 | 1577.0 | 38 | -0+31 | 1576.9 |
| 19 | 0+26.0 | 1577.0 | 39 | -0+32 | 1576.8 |
| 20 | 0+23.0 | 1577.0 | 40 | -0+33 | 1576.7 |



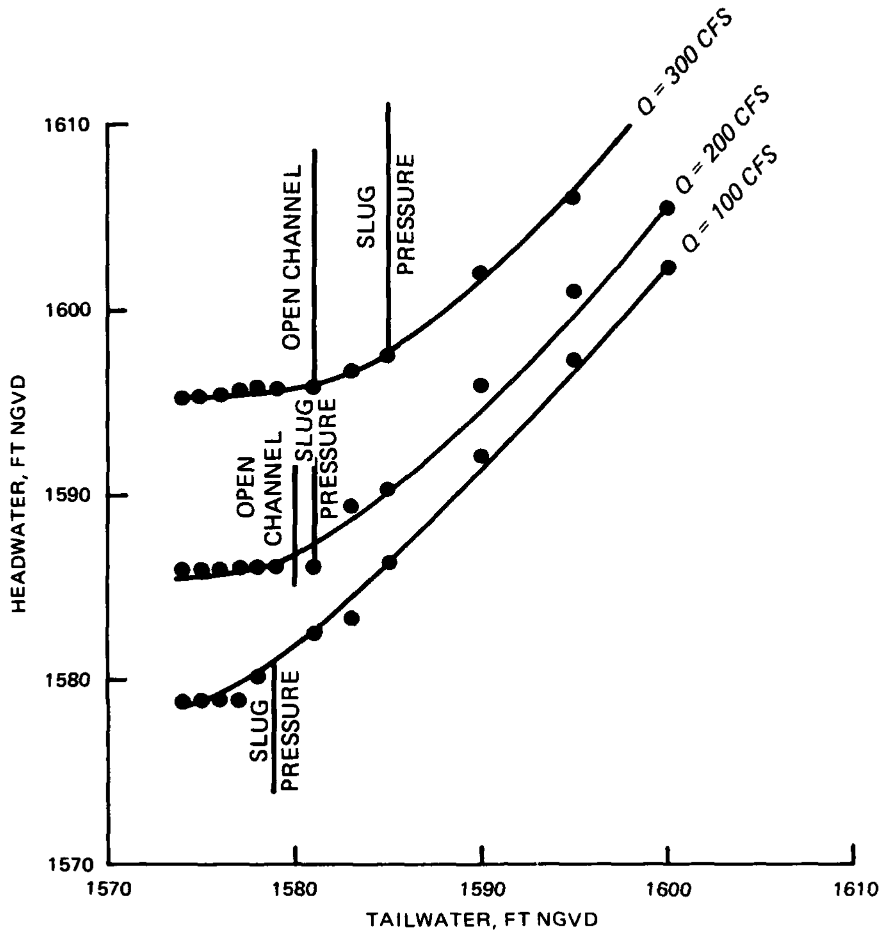

LOCATION OF PIEZOMETER TUBES
SLUICE



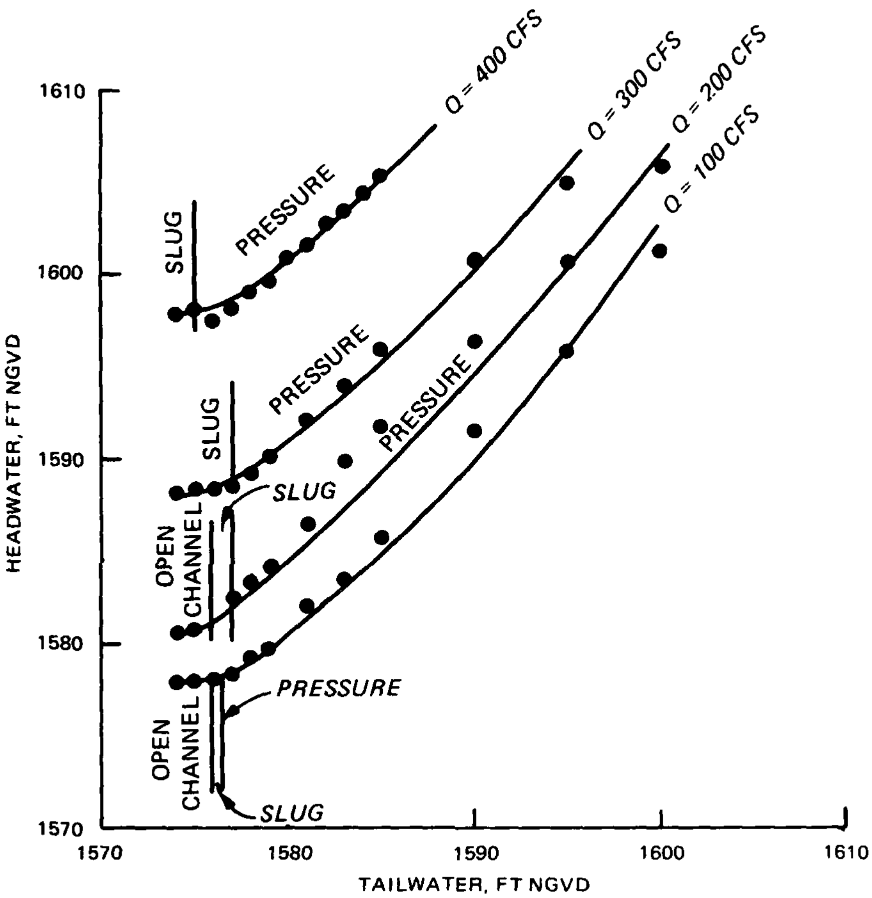

SLUICE
HEADWATER-TAILWATER
CURVES
$G_O = 1.0'$



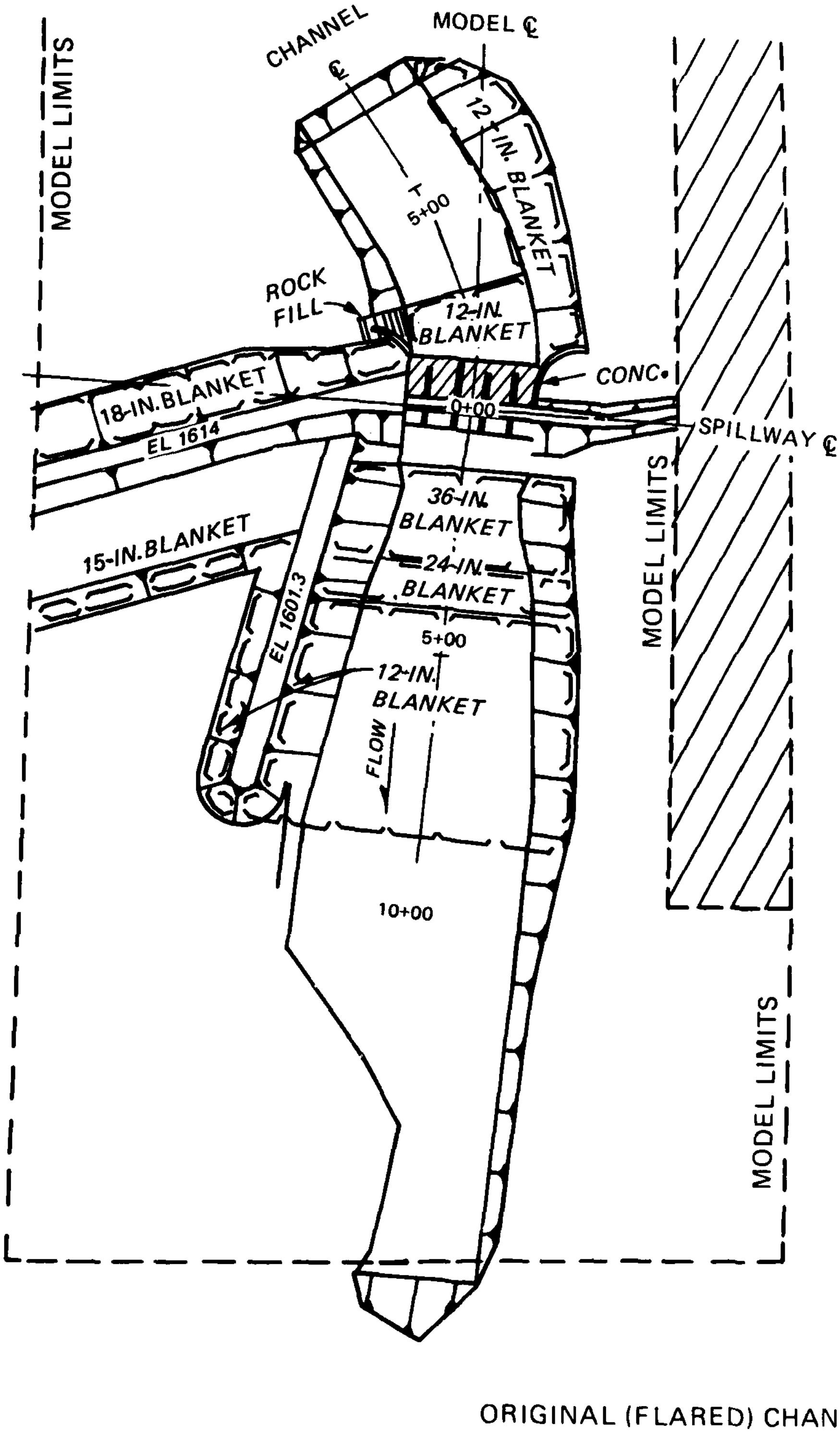

SLUICE
HEADWATER-TAILWATER
CURVES
$G_o$ = 2.0'



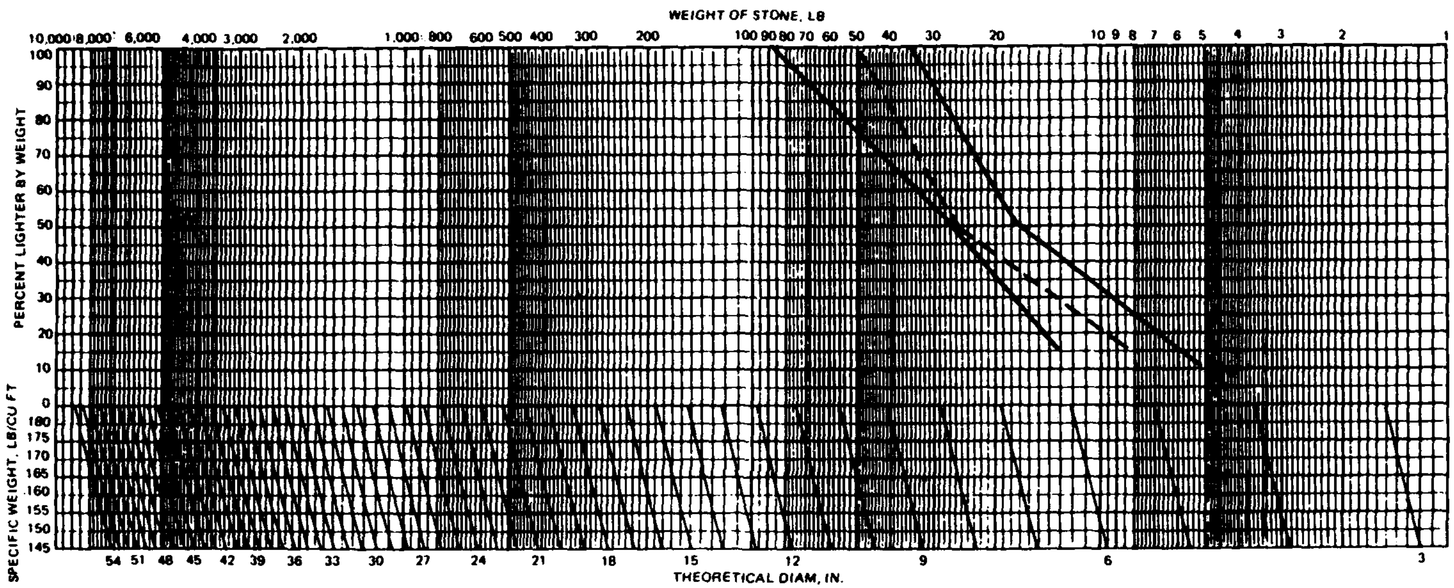

SLUICE
HEADWATER-TAILWATER
CURVES
$G_o = 3.0'$




SLUICE
HEADWATER-TAILWATER
CURVES
$G_O = 4.0'$




ORIGINAL (FLARED) CHANNEL
AND
RIPRAP PROTECTION DETAILS



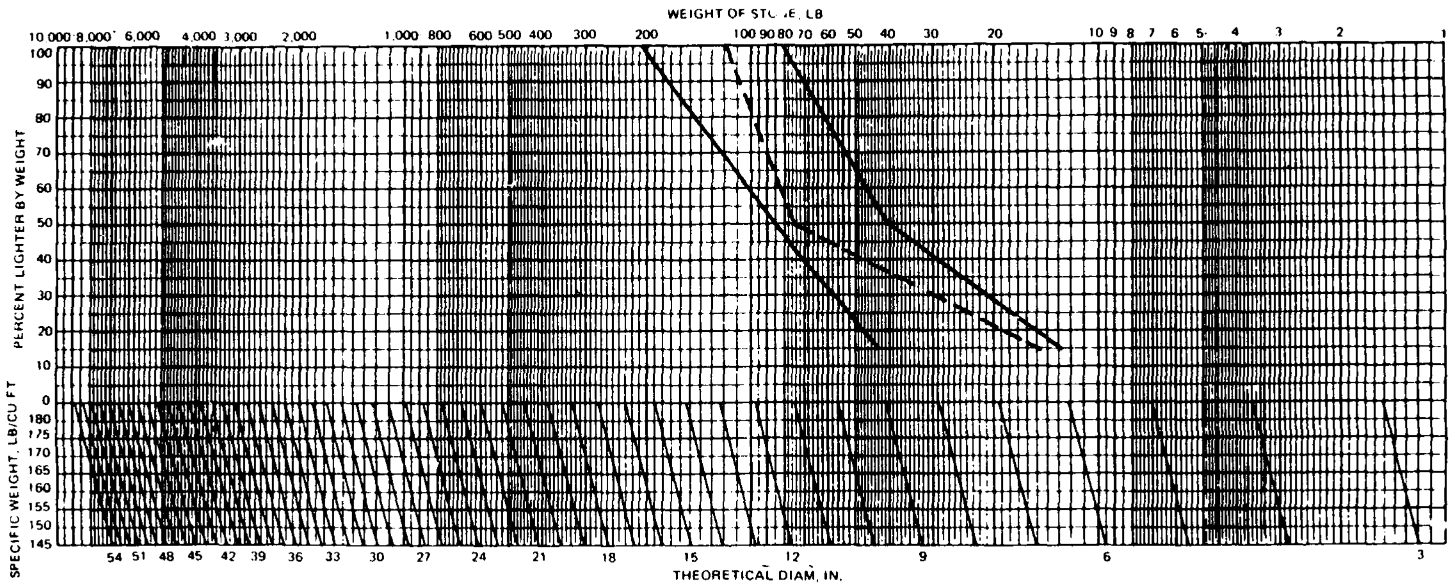

LEGEND

PROTOTYPE GRADATION

MODEL GRADATION

SPECIFIC WEIGHT OF STONE 165 LB/CU FT

RIPRAP GRADATION CURVES
12-IN. THICK BLANKET
15-IN. THICK BLANKET
18-IN. THICK BLANKET




SPECIFIC WEIGHT OF STONE 165 LB/CU FT

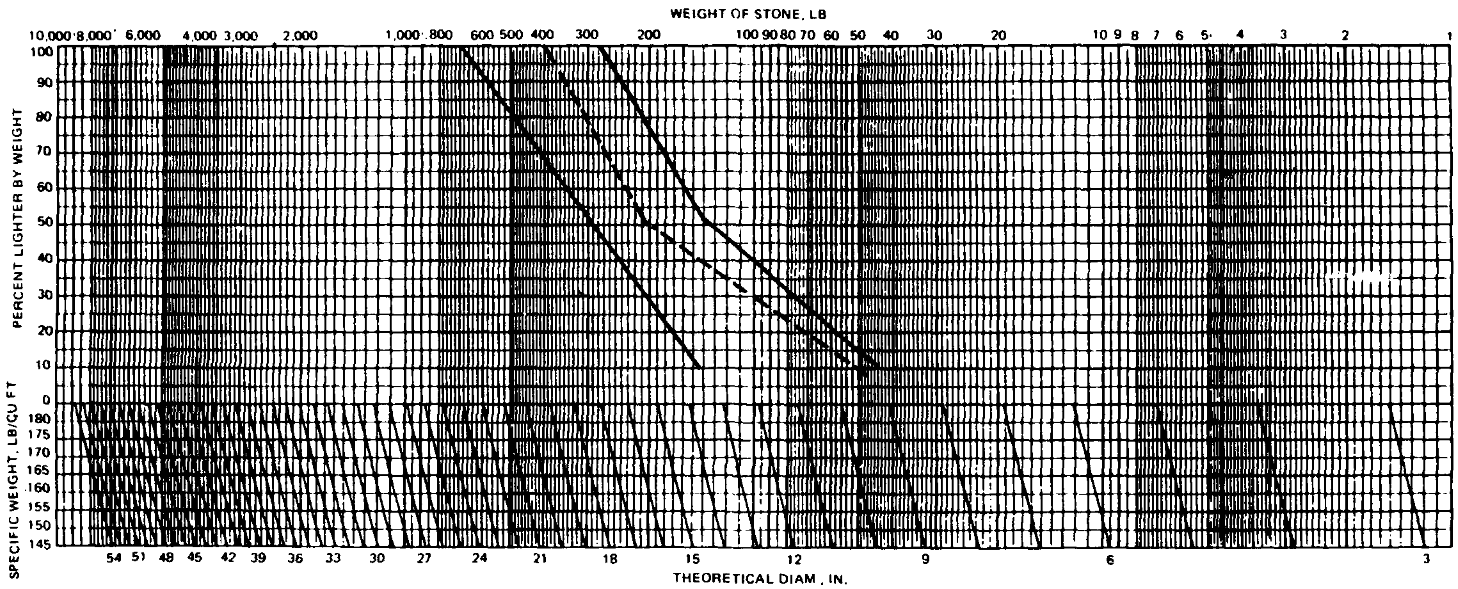
LEGEND

PROTOTYPE GRADATION

MODEL GRADATION

RIPRAP GRADATION CURVES

24-IN.-THICK BLANKET

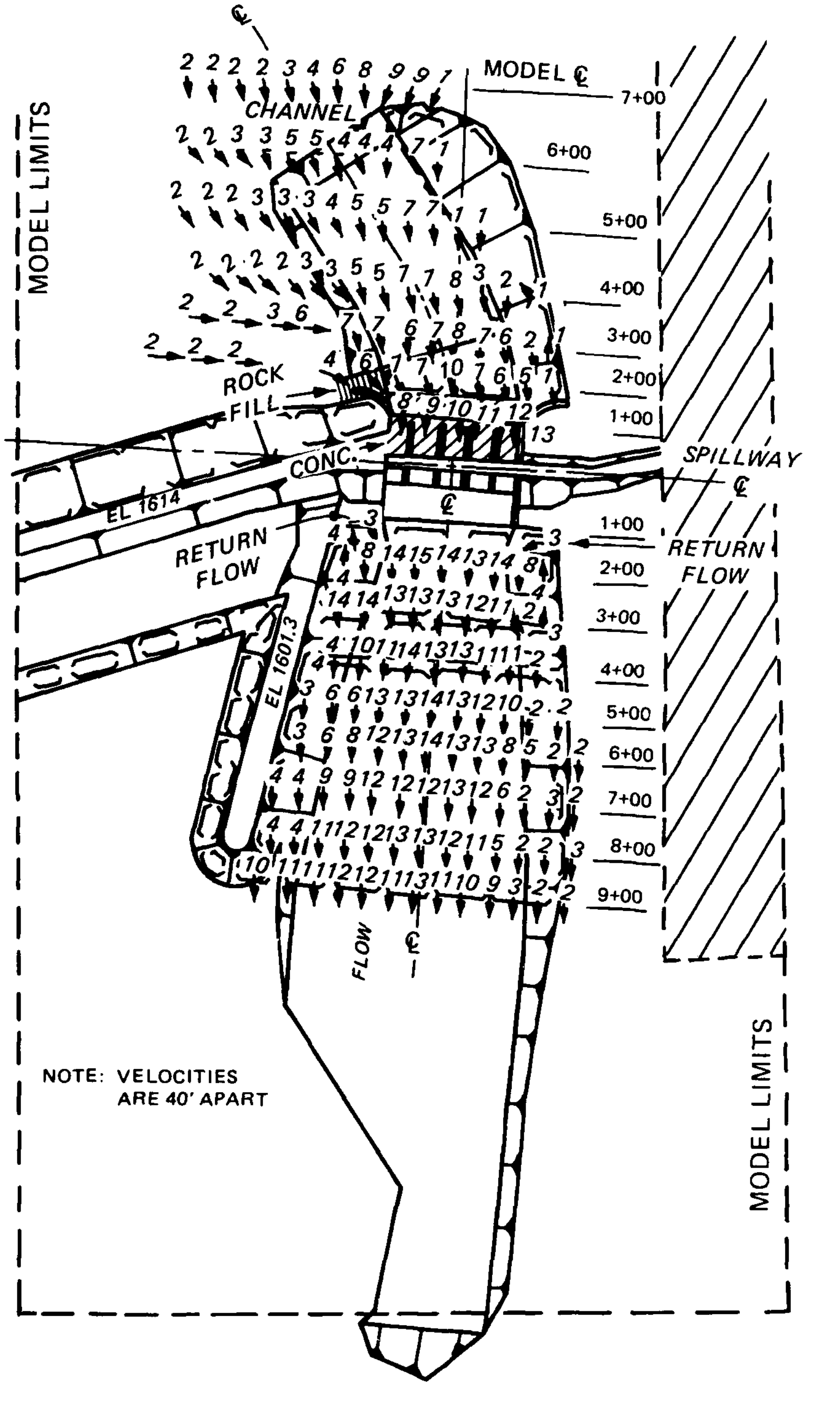



LEGEND

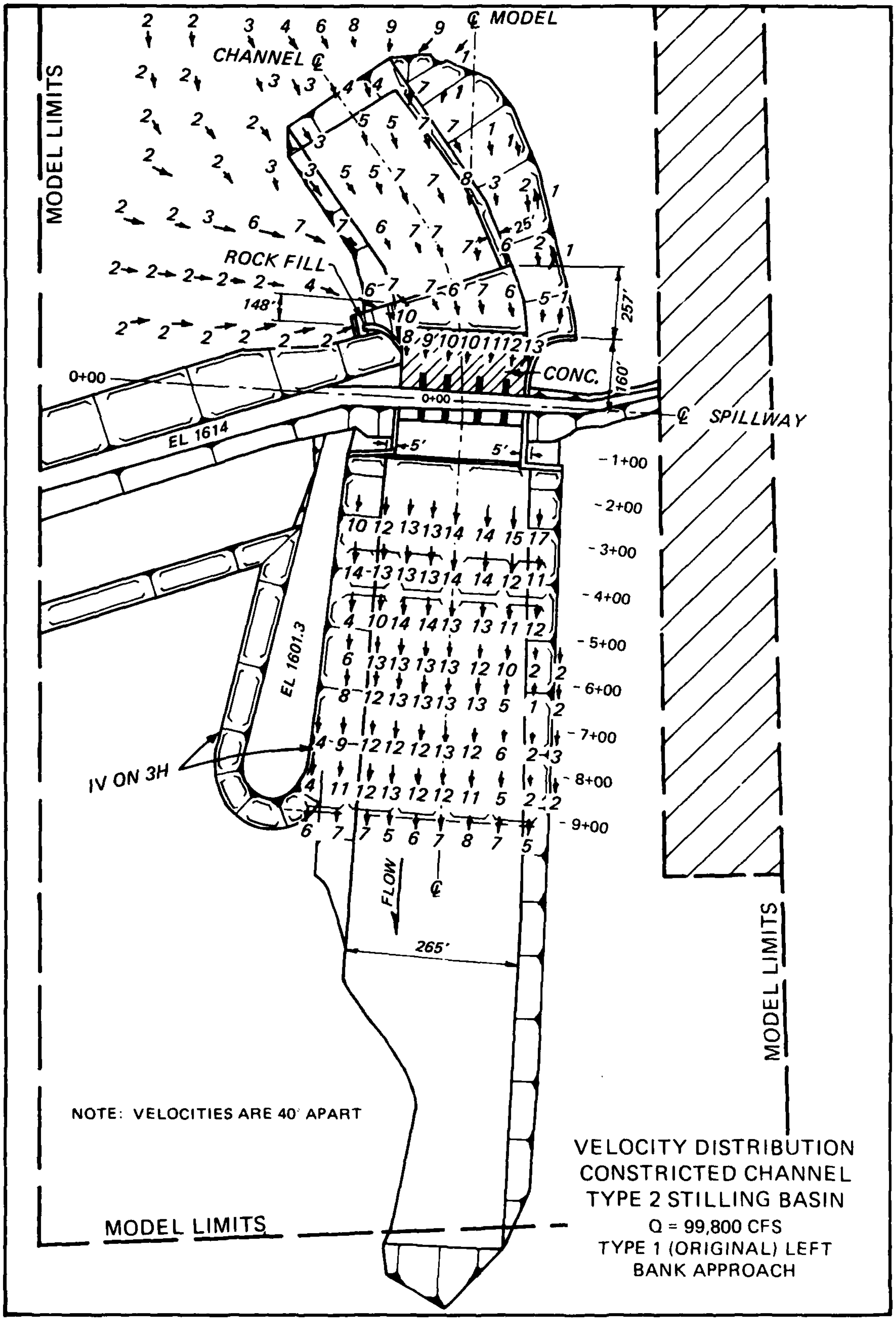
PROTOTYPE GRADATION
MODEL GRADATION

SPECIFIC WEIGHT OF STONE 165 LB/CU FT

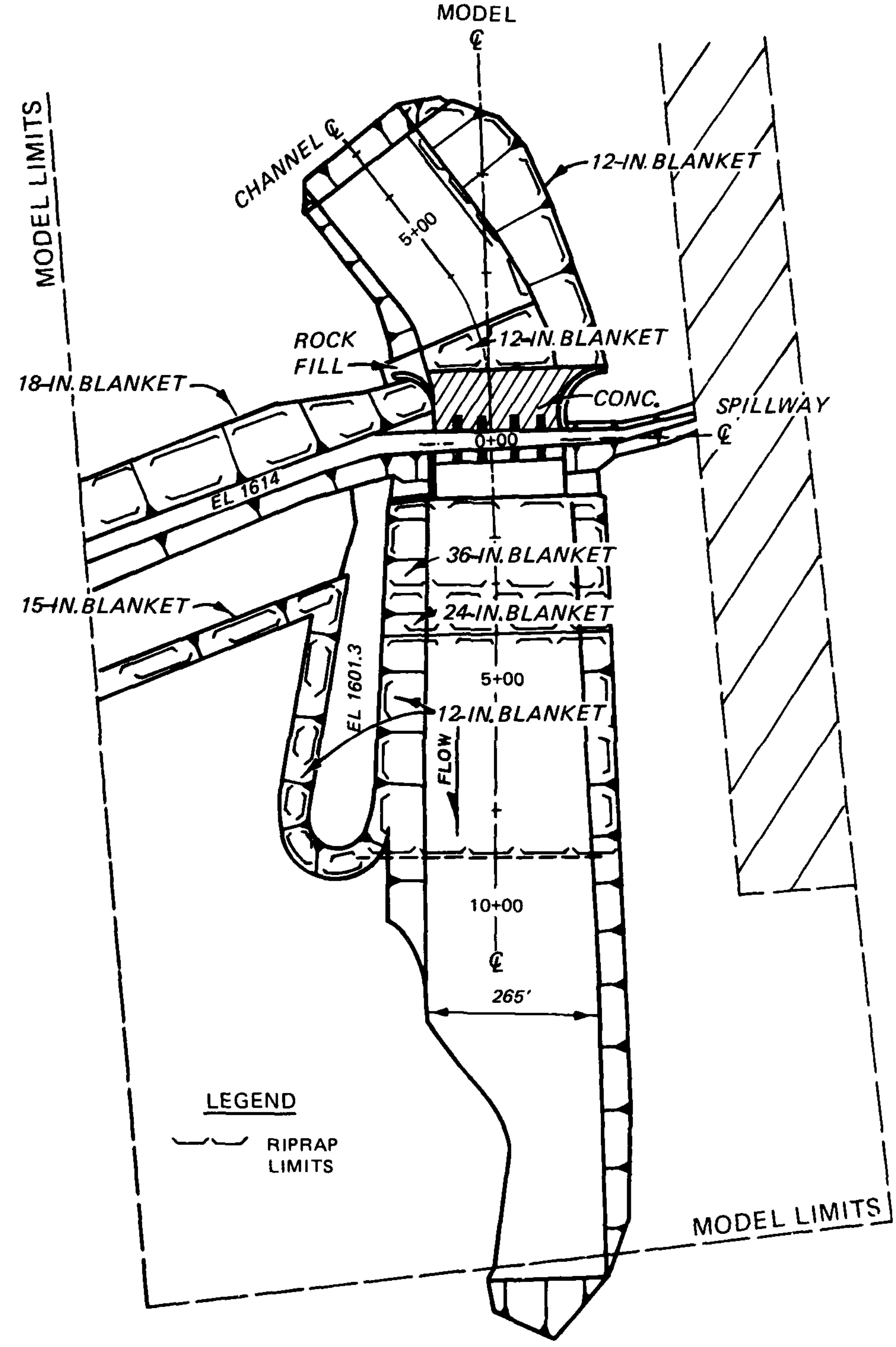
RIPRAP GRADATION CURVES
36-IN. THICK BLANKET



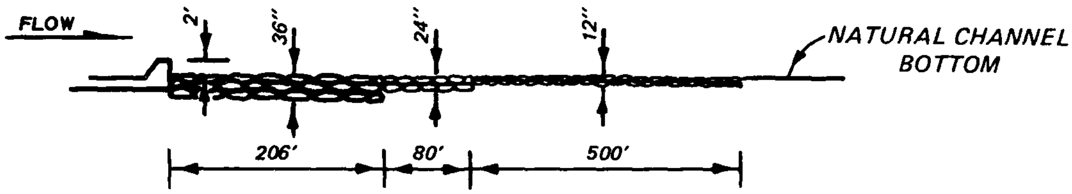

VELOCITY DISTRIBUTION
ORIGINAL CHANNEL
TYPE I STILLING BASIN
Q = 99,800 CFS



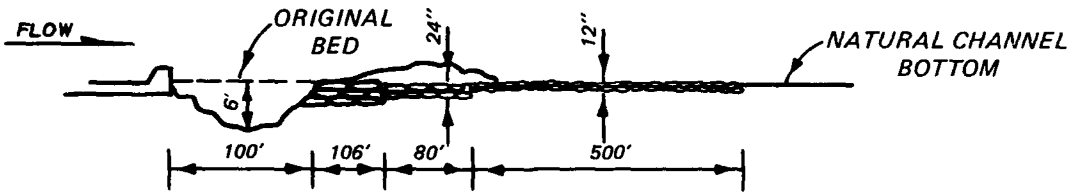

VELOCITY DISTRIBUTION
CONSTRICTED CHANNEL
TYPE 2 STILLING BASIN
Q = 99,800 CFS
TYPE 1 (ORIGINAL) LEFT
BANK APPROACH



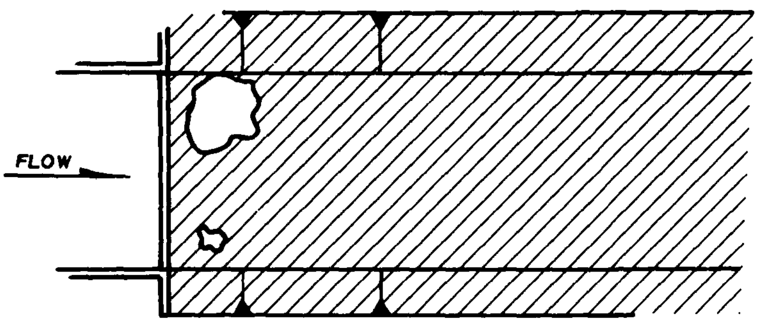

CONSTRICTED CHANNEL
AND RIPRAP PROTECTION DETAILS
TYPE 1 (ORIGINAL)
LEFT BANK APPROACH




RIPRAP PROFILE BEFORE PMF
T = 0 HR



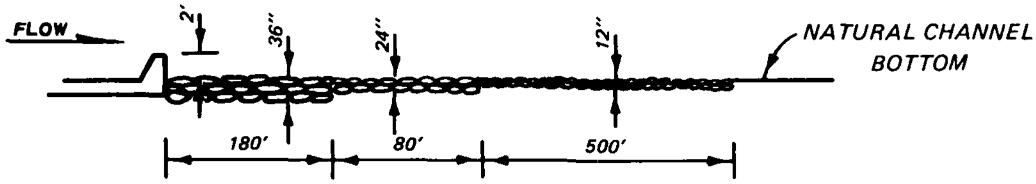

RIPRAP PROFILE AFTER PMF
T = 18 HR



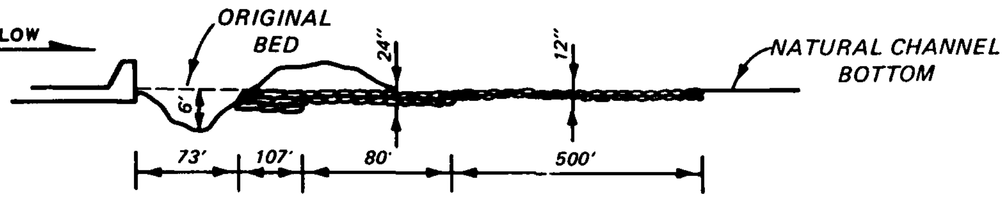

PLAN VIEW AFTER PMF

NOTE: PMF: Q = 99,800 CFS

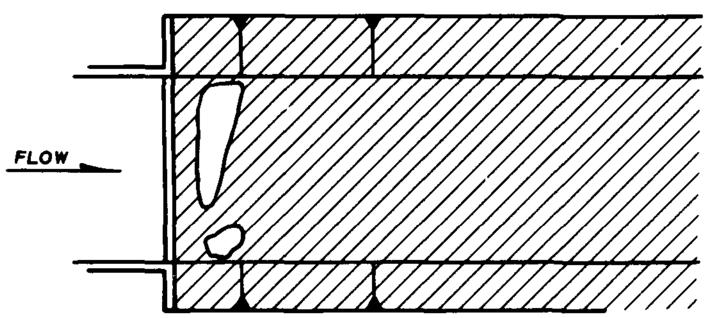
TYPE 1 (ORIGINAL)
RIPRAP PROTECTION
TYPE 1 STILLING BASIN
TYPE 1 (ORIGINAL) LEFT BANK APPROACH




RIPRAP PROFILE BEFORE PMF
T = 0 HR



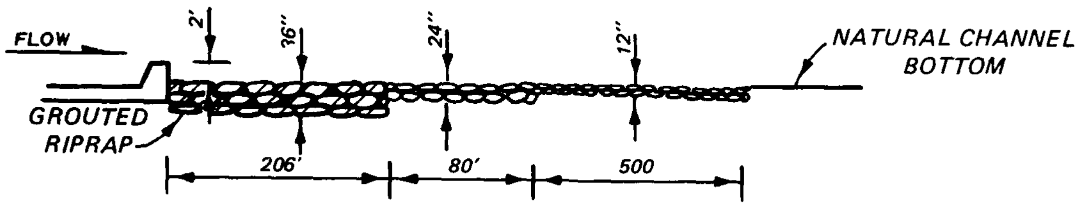

RIPRAP PROFILE AFTER PMF
T = 18 HR



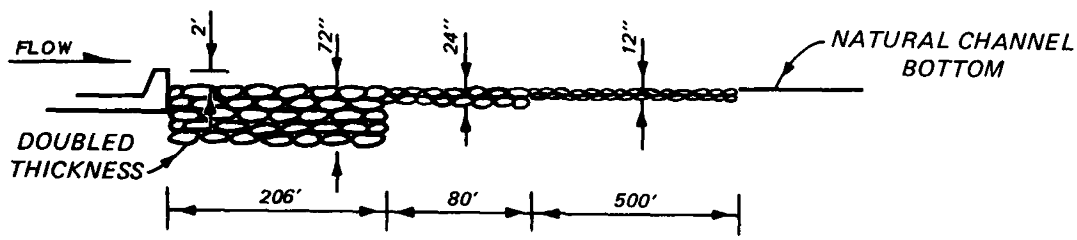

PLAN VIEW AFTER PMF

NOTE: PMF: Q = 99,800 CFS

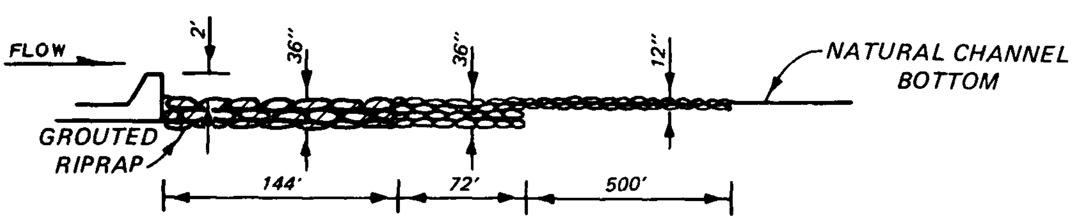
TYPE 1 (ORIGINAL)
RIPRAP PROTECTION
TYPE 2 STILLING BASIN
TYPE 1 (ORIGINAL) LEFT BANK APPROACH




TYPE 2 (GROUTED) RIPRAP
(TYPE 1 BASIN)

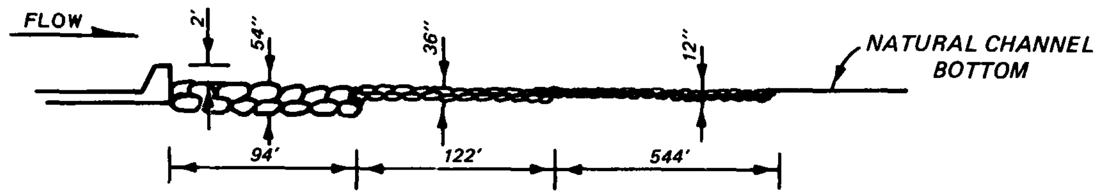



TYPE 3 RIPRAP

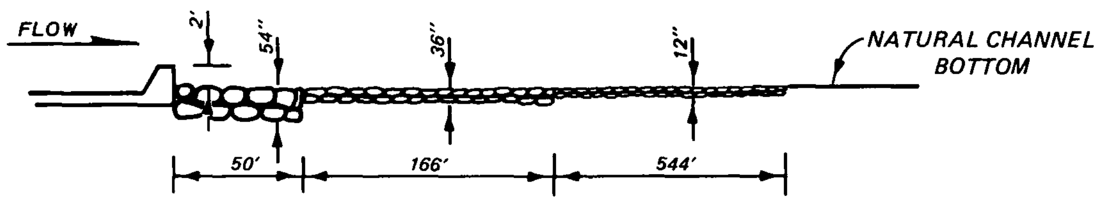



TYPE 4 (GROUTED) RIPRAP
(TYPE 2 BASIN)

TYPES 2, 3, AND 4
RIPRAP PROTECTION
TYPE 1 (ORIGINAL) LEFT
BANK APPROACH



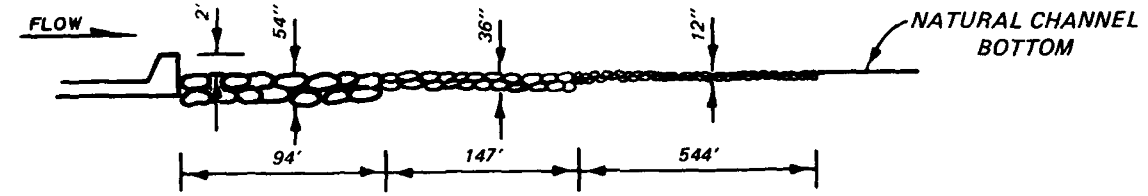

TYPE 5 RIPRAP



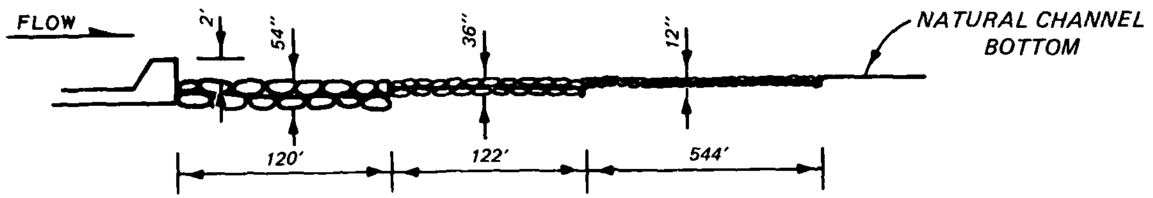

TYPE 6 RIPRAP

NOTE: 54" IS UNIFORMLY GRADED ROCK 2-3 FT IN DIAM.

TYPES 5 AND 6
RIPRAP PROTECTION
TYPE 2 STILLING BASIN
TYPE 1 (ORIGINAL)LEFT
BANK APPROACH




TYPE 7 RIPRAP




TYPE 8 RIPRAP

NOTE: 54" IS UNIFORMLY GRADED ROCK 2-3 FT IN DIAM.

TYPES 7 AND 8
RIPRAP PROTECTION
TYPE 1 STILLING BASIN
TYPE 1 (ORIGINAL) LEFT
BANK APPROACH