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HYDRAULIC MODEL STUDY OF RIGHT AUXILIARY SPILLWAY AT STEWART MOUNTAIN DAM

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16. ABSTRACT

Hydraulic model studies were conducted to evaluate the proposed design for the right auxiliary spillway at Stewart Mountain Dam. The new spillway on the right abutment will be used with the existing left abutment spillway to pass the increased design discharge. The model investigated the spillway approach flow conditions, discharge capacity, flip bucket design, and energy dissipation characteristics of the plunge pool. The primary concern was the possibility of erosion damage in the plunge pool, which could endanger the spillway structure. The flip bucket exit angle was increased and the plunge pool was modified. This improved the energy dissipation in the plunge pool and the flow distribution in the downstream river channel.

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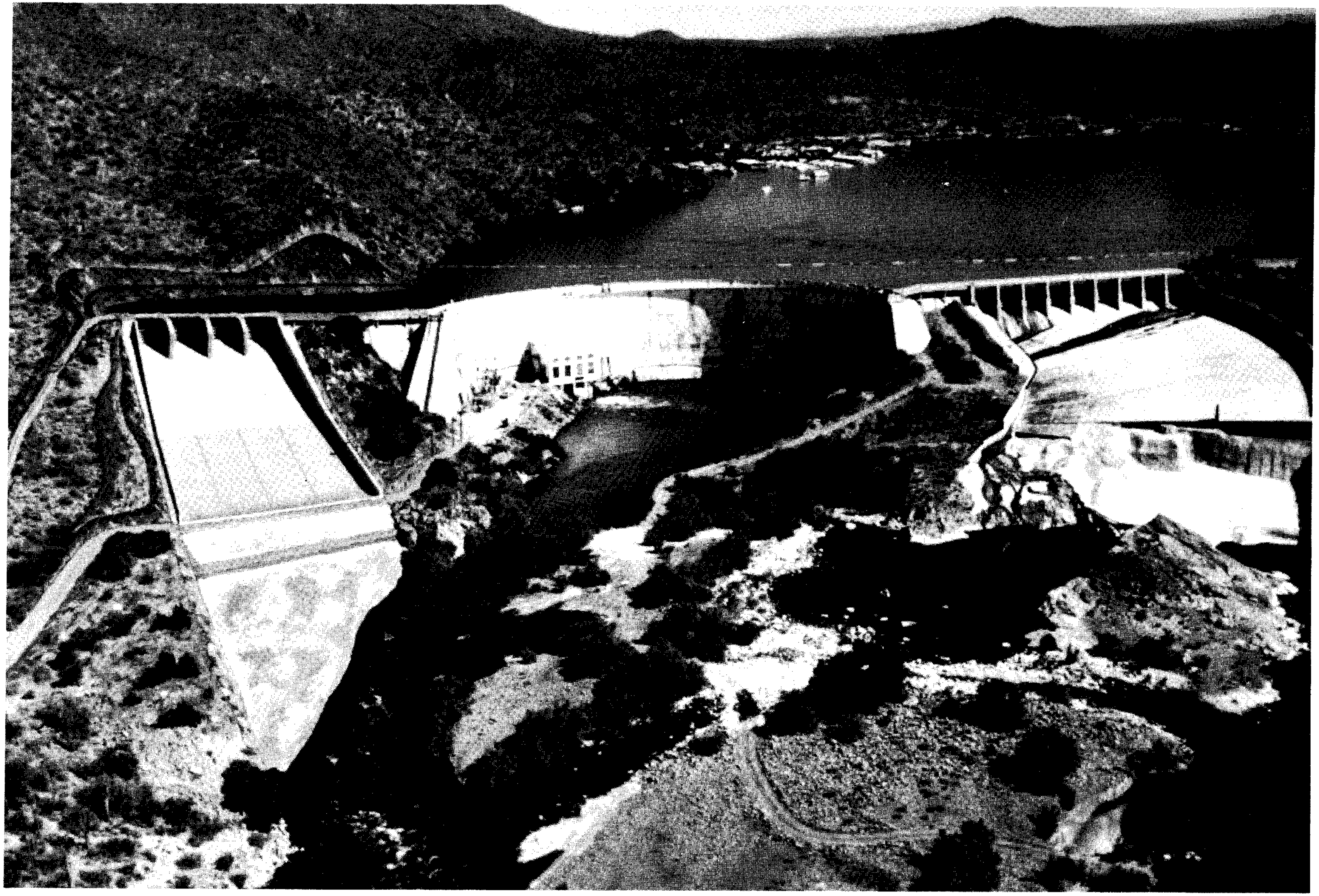
Rotation engineer, David Woodward, greatly assisted with gathering model data. Thomas J. Rhone, Head, Hydraulic Structures Section, contributed to the final design concepts. Photographs were taken by Wayne Lambert.



Frontispiece. – Artist's conception of the existing and the proposed auxiliary spillways at Stewart Mountain Dam.

CONTENTS

TABLE

FIGURES

CONTENTS – Continued

INTRODUCTION

Stewart Mountain Dam is located on the Salt River, about 40 miles northeast of Phoenix, Arizona. The dam was completed in 1930, and is operated by the SRP (Salt River Project). The major features of the project are a 200-foot-high dam with a radial gate-controlled, superelevated spillway on the left abutment, outlet works, and a powerplant.

An evaluation of Stewart Mountain Dam was completed under the SEED (Safety Evaluation of Existing Dams) program. Several unsafe conditions were discovered, including the inability to pass the PMF (probable maximum flood) without overtopping the dam. Extensive rehabilitation of existing hydraulic structures and the addition of an auxiliary spillway, jointly funded by the Bureau (Bureau of Reclamation) and the SRP, are planned to correct the identified deficiencies.

PURPOSE

This report documents the results of the hydraulic model study used to evaluate the proposed design of the right abutment auxiliary spillway. This spillway together with the rehabilitated existing spillway were designed to pass the PMF. The proposed auxiliary spillway is shown in plan and section on figure 1. The model investigated the following features:

- Spillway approach channel configuration and flow velocity distribution patterns
- Spillway discharge capacity, chute water surface profiles, and unequal gate operations
- Characteristics of the spillway flip bucket
- Plunge pool configuration
- Potential for erosion downstream of the spillway, determined by velocity and pressure measurements

RESULTS

The following results and recommendations are made based upon the model study:

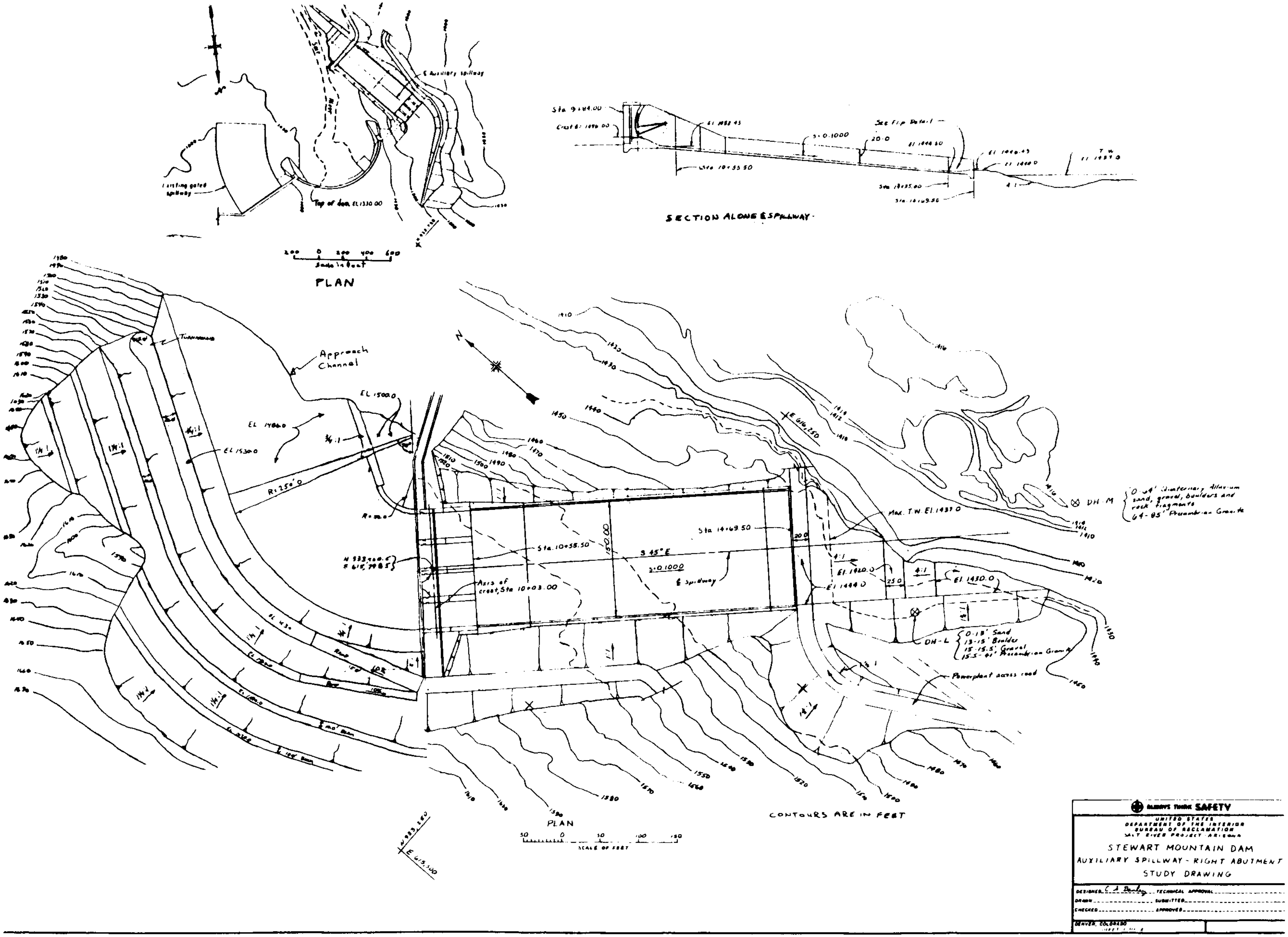


Figure 1. – Plan and section of initial design for right auxiliary spillway at Stewart Mountain Dam.

Approach Channel. – Construct a vertical semicircular guidewall to El. 1532.0 feet that extends from the left side of the spillway entrance into the reservoir. This wall will improve the flow conditions upstream of the left spillway bay, thus increasing spillway capacity. A vertical wall should be placed between the far right pier and the embankment cut slopes that form the right side of the approach channel. The recommended approach channel configuration is shown on figure 2.

The existing reservoir stilling well is located on the face of the dam near the right thrust block. This location will be upstream of the right spillway guidewall and the topography that extends into the reservoir near the left spillway bay. Because it is behind the guidewall, the well should be sufficiently removed from the effects of drawdown produced by operation of the right spillway. However, the optimum location would be the deepest portion of the reservoir, about the mid-point of the dam.

Discharge Capacity. – At maximum reservoir El. 1532.0, the discharge will be 94,000 ft^3/s through the auxiliary spillway with the gates fully open. This exceeded the required design discharge of 89,000 ft^3/s. Discharge curves were developed for equal gate operation in 3-foot gate opening increments (fig. 3).

Gate Operation. – Uniform gate operation is recommended because it produces the best flow conditions in the spillway chute and plunge pool downstream. Nonuniform gate operation was investigated, but should only be used during an emergency.

Flip Bucket. – Two flip buckets were tested: the initial 15-degree (above horizontal) bucket and the recommended 35-degree bucket. The 35-degree bucket (fig. 4), formed by a 45-foot radius beginning at Sta. 14+37, will provide better energy dissipation in the plunge pool and more uniform flow conditions in the plunge pool and downstream river channel.

Initial opening of the gates will cause a hydraulic jump to form in the chute upstream of the flip bucket. Until sweepout occurs at a discharge of about 6,000 ft^3/s, flow over the end of the flip bucket will impinge on the powerplant road.

Chute Wall Heights. – The chute sidewalls must contain flow depths associated with maximum discharge and the hydraulic jump upstream of the 35-degree flip bucket before sweepout. Flow depths were measured along the chute wall normal to the slope. Flow depths for maximum discharge (94,000 ft^3/s) were:

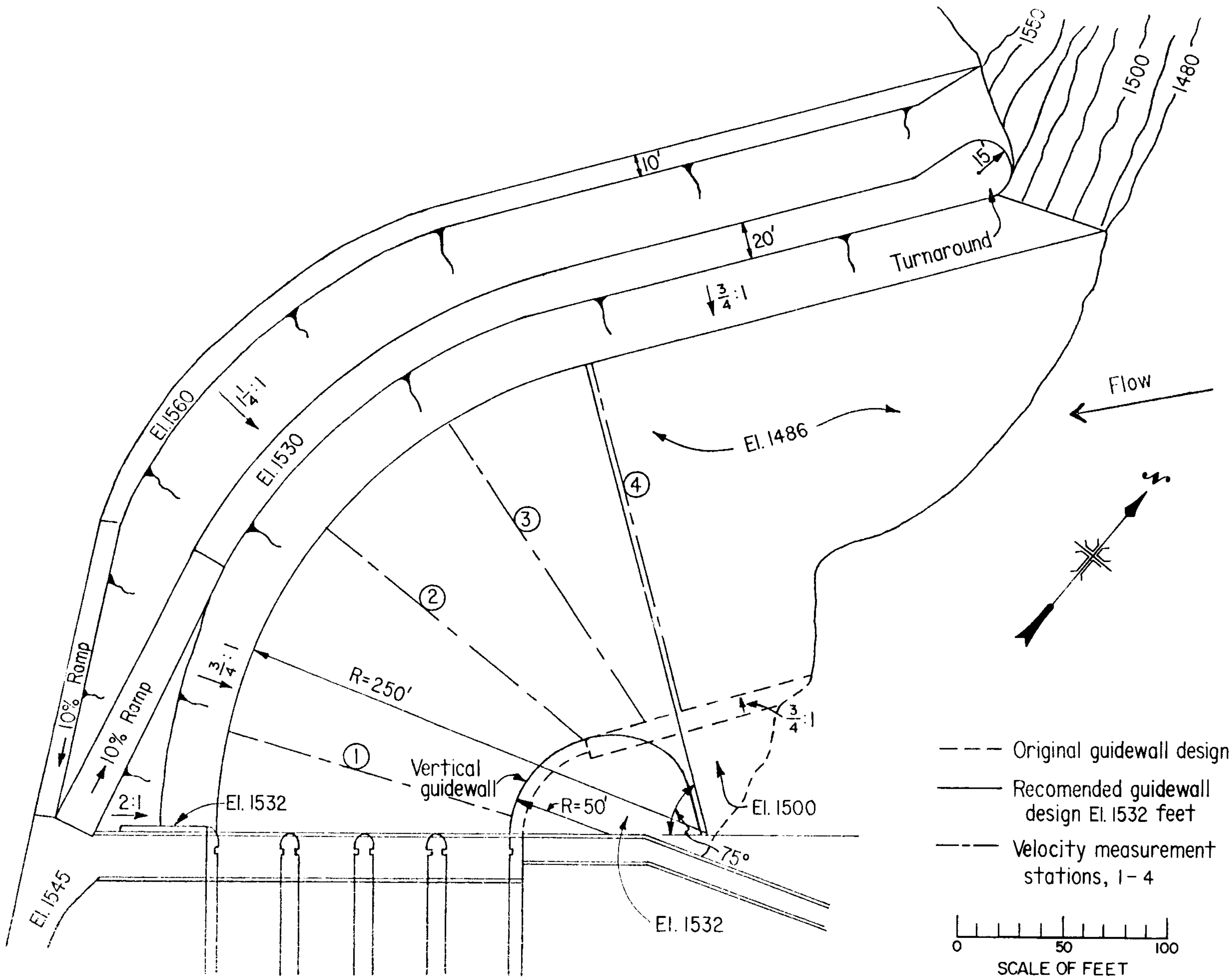


Figure 2. – Approach channel geometry and velocity measurement stations with original and recommended designs.

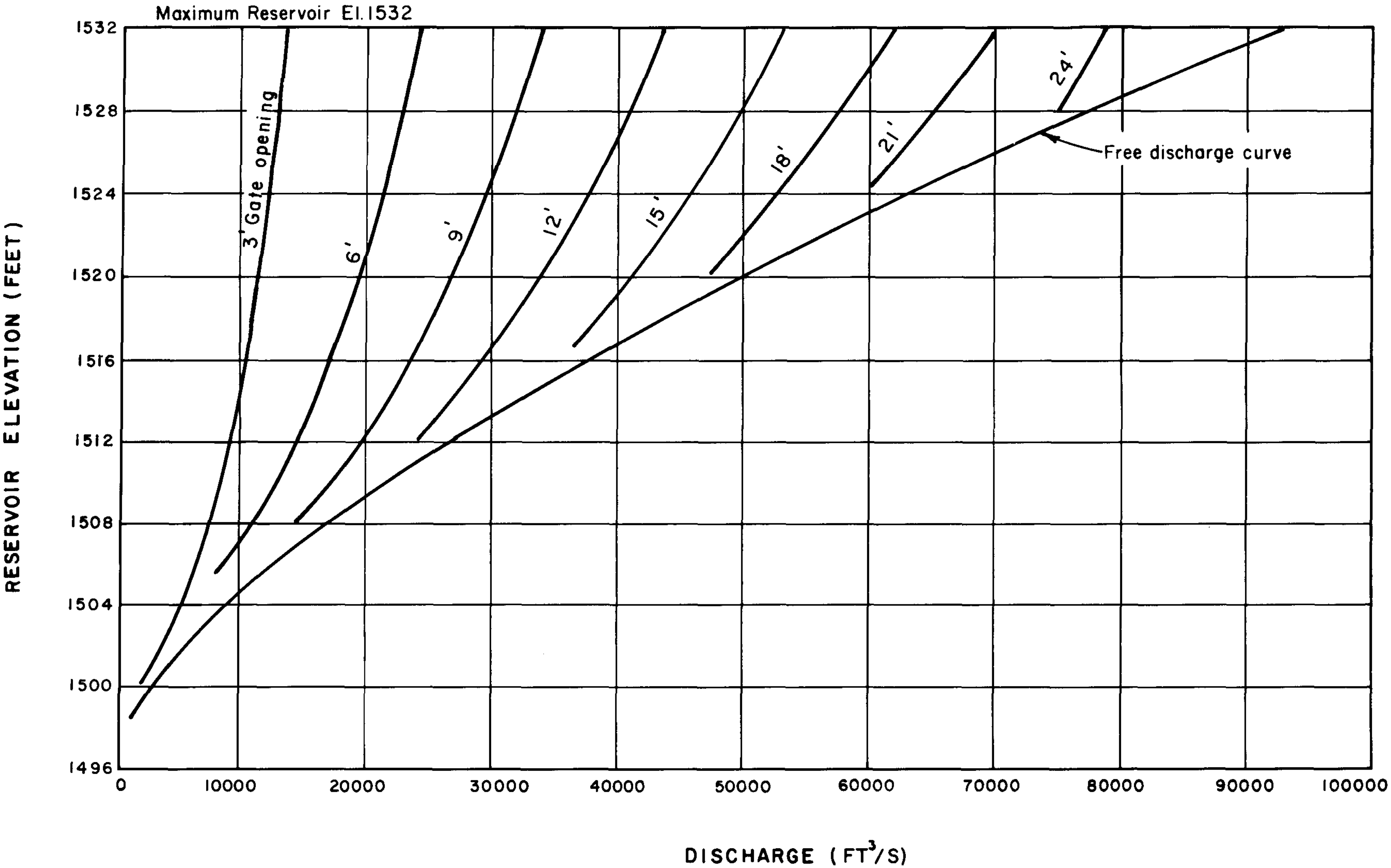


Figure 3. – Right auxiliary spillway discharge curves for 3-foot gate opening increments.

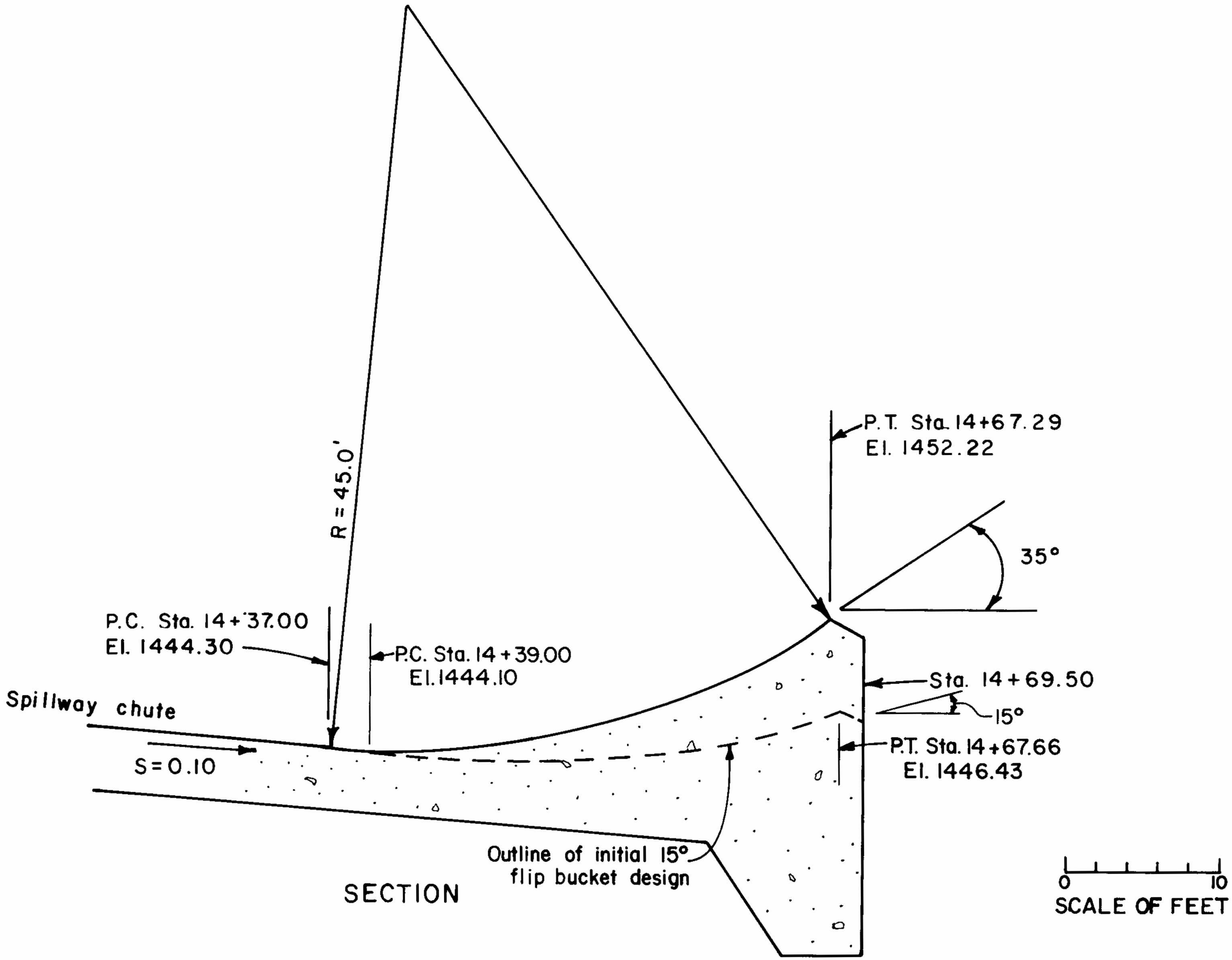


Figure 4. – Recommended flip bucket design.

Location	*Depth*
Sta. 11+57.5	13.5 feet
Sta. 14+05	12.9 feet
Sta. 14+67.29	17.5 feet

Flow depths immediately upstream of the flip bucket were greater with the hydraulic jump in the chute than when passing maximum discharge. Before sweepout, the water surface upstream of the flip bucket will follow the shape of a hydraulic jump in a steep channel. The chute walls should contain a flow depth of 13.5 feet at Sta. 14+05 and slope upward to contain a 20-foot depth at Sta. 14+45. The water surface profiles for maximum discharge and before sweepout are shown on figure 5. The flow depth before sweepout requires raising the wall heights at the flip bucket.

Basin Excavation. – The upstream boundary of the plunge pool from the powerplant road (El. 1444.0) to the pool floor (El. 1410.0), should be excavated to a 1.5:1 slope. The right bank of the plunge pool should be cut on a 0.75:1 slope from the floor to El. 1450.0, continuing downstream for about 250 feet where the cut slope will meet the existing topography. The left side of the plunge pool is formed by the river channel. Velocities downstream of the basin for maximum discharge were consistently in the upper 30 ft/s range, but were evenly distributed (fig. 23). This pool configuration (fig. 6) will provide the best flow distribution in the pool and river channel downstream.

The proposed location for the helicopter pad on the right hillside adjacent to the basin will experience a great deal of spray during medium- to high-range releases. Occasionally, waves will overtop the pad during maximum discharge.

THE HYDRAULIC MODEL

A linear scale of 1:40 was chosen for the model. This scale allowed adequate modeling and investigation of the necessary prototype hydraulic features. The existing spillway on the left abutment was not modeled. It was determined that energy dissipation below the auxiliary spillway would be adequately modeled without supplying discharges from the left spillway, provided tailwater depths representing both spillways in operation were modeled.

The model included 600 feet of the reservoir and spillway approach channel, the 150-foot-wide right auxiliary spillway chute, and about 600 feet of the downstream river channel. The reservoir

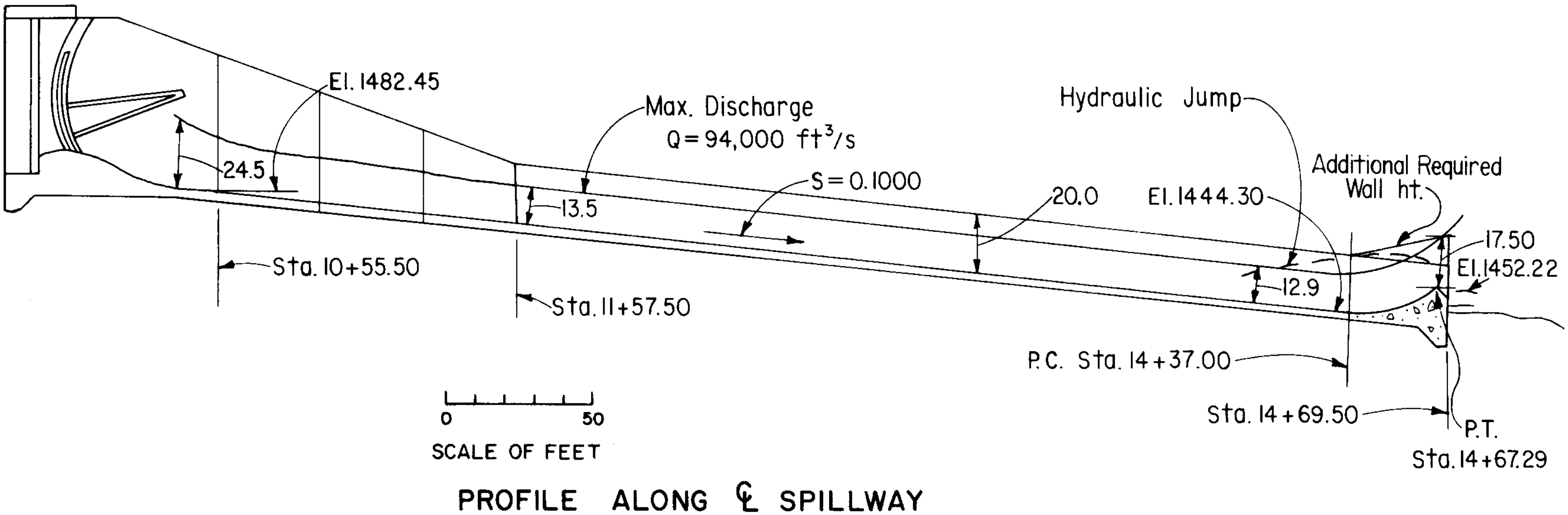


Figure 5. – Chute water surface profile before sweepout for Q = 94,000 ft³/s.

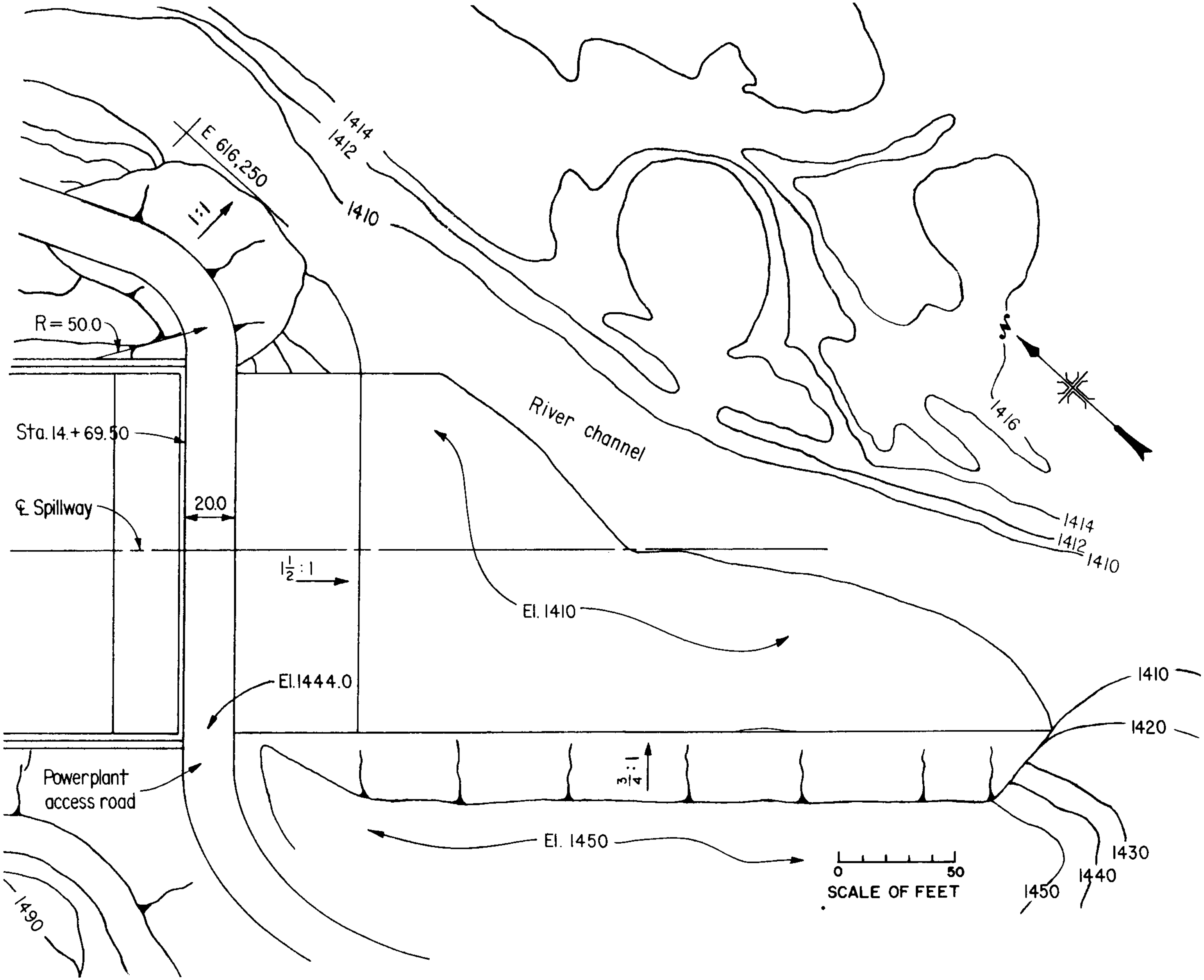


Figure 6. – Recommended plunge pool design.

and spillway intake channel were modeled inside a 15- by 30-foot headbox. The radial gate-controlled auxiliary spillway was modeled by a 3.75-foot-wide spillway crest with four 0.75- by 0.825-foot radial gates, and a 10.35-foot-long chute and flip bucket. The river channel was modeled for 15 feet downstream of the auxiliary spillway.

The model was sized to allow investigation of flow patterns across the entire width of the downstream channel. Topography was modeled for the river channel, an area extending 320 feet to the left of the plunge pool, and for the hillside to El. 1450, right of the channel. Operation of the model showed that the area modeled was adequate, because all excessive wave heights or velocities dissipated within the modeled area.

An overall view of the 1:40 scale model is shown on figure 7.

SPILLWAY OPERATING CRITERIA

The existing spillway on the left abutment will be used as the service spillway, providing tailwater that will assist energy dissipation of discharges from the auxiliary spillway. Initial investigations were conducted under operating criteria that required the existing spillway to pass 120,000 ft^3/s (TW (tailwater) El. 1434.8) before operating the auxiliary spillway. Therefore, substantial tailwater depth was available for energy dissipation before operation of the auxiliary spillway. Most model testing was completed under these criteria; however, the operating criteria were changed near the end of the model testing. The final criteria require the existing spillway to pass 75,000 ft^3/s (TW El. 1430.0), then to add 94,000 ft^3/s (TW El. 1439.4) from the auxiliary spillway before increasing the discharge of the existing spillway to 120,000 ft^3/s. This produced a combined total spillway discharge of 214,000 ft^3/s at reservoir El. 1532.0 and TW El. 1443.3. The auxiliary spillway reached maximum discharge under these operating criteria with the tailwater elevation about 3.9 feet lower than the original criteria. The recommended plunge pool design was evaluated under the tailwater conditions produced by these final operating criteria.

INVESTIGATION

Approach Channel

Proper alignment of the auxiliary spillway with the river channel downstream will require extensive excavation of an upstream approach channel. Approach channel geometry will require excavation

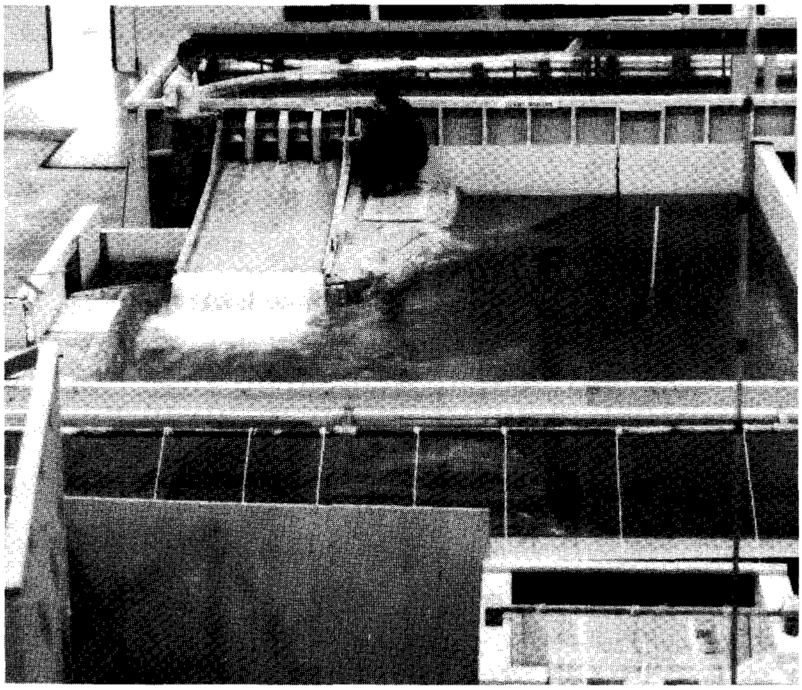


Figure 7. – Stewart Mountain Dam auxiliary spillway, 1:40 scale model.
P801-D-81058.

of a 150-foot-wide channel to El. 1486.0, 10 feet below the spillway crest elevation of 1496.0 (fig. 1). The channel will be excavated on a 250-foot radius, producing an abrupt 75-degree turn from the reservoir to the spillway entrance. The cut slope from the floor of the channel along the right side will be 0.75:1 up to a 20-foot-wide berm at El. 1530.0. Above El. 1530.0, the cut slope will be 1.25:1 with a berm every 30-foot rise in elevation. The above portion of the approach channel geometry was not modified during the model investigations. For the initial design, the left side of the channel was excavated to a vertical, 50-foot-radius circular section, 14 feet high, from the left side of the spillway entrance into the reservoir at El. 1500.0. This was extended farther into the reservoir along the face of the dam by a 0.75:1 slope from the channel floor to El. 1500.0 (original design, fig. 2). Maximum water surface elevation is 1532.0.

Initial model operation revealed high-velocity flow along the face of the dam over the topography at El. 1500.0. As this flow, perpendicular to the spillway crest centerline, abruptly turned to enter the left spillway bay, a large contraction formed at the end pier. The crest section immediately adjacent to the pier did not pass any flow; this significantly reduced the discharge.

Several guidewall arrangements were investigated to reduce the contraction and, thus, increase the discharge capacity. The optimum solution was to increase the height of the original vertical guidewall formed by the 50-foot radius to maximum reservoir El. 1532.0. This will require less excavation of the original ground surface in the prototype, but stabilization with a concrete wall. This wall should then extend to the left along the face of the dam for about another 50 feet to almost form a semicircle. This significantly improved flow through the left bay, will increase the total prototype spillway discharge by about 4,000 ft^3/s. The recommended design for the guidewall left of the spillway entrance is shown in plan on figure 2.

The initial approach channel geometry on the right side near the dam abruptly changed from the 0.75:1 slope to the vertical faces of the dam and spillway end pier. The spillway pier also partially extended upstream of the face of the dam into the reservoir. This geometry produced a slight contraction at the end pier during higher discharges.

The right side of the approach channel was modified by installing a warped surface from the 0.75:1 cut slope to the vertical pier. This configuration produced the best flow condition, but because the warped surface would be expensive to construct in the prototype, the final recommendation was to construct a vertical wall parallel to the dam from the upstream pier nose to the excavated rock slope.

Velocities were measured at four stations in the approach channel of the model (fig. 2). Measurements were recorded at the base of the 0.75:1 slope, the centerline of the channel, and about 10 feet from the left guidewall, each at 0.2, 0.5, and 0.8 of the approach channel depth. The velocities increased from right to left across the channel and as the flow approached the spillway. The velocities were generally low except near the left guidewall.

Average velocities 10 feet from the left guidewall measured at reservoir El. 1543.0 were as follows:

	Velocity (ft/s)	
Discharge (ft^3/s)	*Measurement Station 1*	*Measurement Station 4*
13,000	2.5	1.2
53,000	11.3	4.5
94,000	22.6	8.4

These higher velocities near the left guidewall were expected as a result of the flow phenomena created by the channel bend. The geometry of the bend produced a rise in the water surface along the outer (right) bank and a decrease in the water surface elevation near the inside of the bend, or left side of the approach channel. The water surface and velocities on the inside of the bend were also affected by drawdown associated with spillway operation. During operation, the proximity of the guidewall to the left spillway bay caused an increase in velocity along the guidewall. The high velocities near the left bay will require a concrete guidewall or extensive slope armoring to protect this area from damage.

Discharge Capacity

Flow through the 150-foot-wide spillway was controlled by four 30- by 33-foot radial gates atop a low ogee crest followed by a 10:1 sloping chute and flip bucket terminal structure. Discharge rating curves were developed for this spillway. The free flow spillway discharge was 94,000ft^3/s at maximum reservoir elevation 1532.0. Discharge curves were also developed with all four gates open equal amounts in 3-foot increments up to a 24-foot opening (fig. 3). Gate openings were measured from the top of the crest at El. 1496.0.

Spillway discharge capacity was affected by the approach channel geometry, as discussed in the previous section. The effects of the bend and of the velocity component perpendicular to the spillway centerline were transferred through the gates. The velocity component near the left guidewall was still directed toward the right. The flow was directed more parallel to the spillway centerline when moving across the spillway from left to right. This effect was also observed from the direction of the fins downstream of the piers. Fins will form as flow from the gates meet downstream of the piers. The fin downstream from the left pier was directed substantially toward the right, but this effect dissipated across the chute toward the right. These fins will produce significant spray in the prototype, especially for intermediate discharges.

Gate Operation

Uniform gate operation is recommended because it produces the best flow conditions in the spillway chute and in the downstream plunge pool. The gates should always be operated uniformly except during an emergency such as a gate malfunction.

Nonuniform gate operations were investigated by evaluating flow conditions upstream of the gates, in the spillway chute, and in the downstream plunge pool. Of the several gate opening combinations investigated, the following operation provided the best possible flow conditions at reservoir El. 1532.0. The far right gate (looking downstream) should be opened first. This gate may be opened alone until reaching an 18-foot gate opening or about 15,500 ft^3/s. To continue increasing discharge, the far left gate should then be opened until the opening is 18 feet. Greater releases should be accomplished by opening the two center gates simultaneously until they are also open 18 feet. At this point the discharge would be about 62,000 ft³/s, and larger flows must be passed by opening the gates equal amounts.

Chute Water Surface Profile

The water surface profile along the wall was measured (normal to the chute slope) for the maximum discharge and for the recommended 35-degree flip bucket. This profile indicated a maximum flow depth of 24.5 feet at Sta. 10+42, the gate pin location, decreasing to 13.5 feet at Sta. 11+57.5 and to 12.9 feet at Sta. 14+37, the beginning of the flip bucket. The flow depth at the P.T. (point of tangency) of the flip bucket, Sta. 14+67.29, was 17.5 feet (fig. 5).

During the initial opening of the spillway gates, a jump will form in the chute upstream of the flip bucket. The flow depth associated with the hydraulic jump will govern the wall heights at the upstream end of the flip bucket. Before sweepout at about 6,000 ft³/s, the water surface upstream of the flip bucket will follow the shape of a hydraulic jump in a steep channel. The walls should be high enough to contain a flow depth of 13.5 feet at Sta. 14+05 and a depth of 20 feet at Sta. 14+45 (fig. 5).

These flow depths should permit a reduction in the proposed 20-foot wall heights along all of the chute, except at the flip bucket. To contain the flow depths of the hydraulic jump, the wall height should be increased from 20 feet at Sta. 14+37 to the end of the flip bucket, where the 17.5-foot flow depth at maximum discharge requires a 30-foot wall height.

Velocities and cavitation potential were investigated for the entire length of the chute and flip bucket. Maximum discharge and flow rates of about one-fourth, one-half, and three-fourths of maximum discharge were analyzed. The cavitation index, which is a function of the pressure, fluid density, and velocity, is defined as:

$$\sigma = \frac{P_o - P_v}{\rho\left(\frac{V_o^2}{2g}\right)}$$

where:

P_o = reference pressure = atmospheric plus gauge pressure,
P_v = vapor pressure,

ρ = fluid density, and

V_o = fluid velocity at the reference point.

All values of σ, the cavitation index or flow sigma, were greater than 0.2; therefore, cavitation should not occur along the chute or flip bucket. The low cavitation potential will allow the ends of the chute underdrains and the openings for the flip bucket drains to be left uncovered. The flip bucket drains will not require eyebrows, provided the ratio of the vertical depth of the drain opening to the drain diameter is greater than or equal to one. The results of the cavitation analysis are shown on figure 8.

Potential for River Channel Erosion

Historically, erosion damage has been a problem below the existing spillway at Stewart Mountain Dam. Flow conditions in the impact area downstream of the auxiliary spillway were studied to prevent erosion damage, which may endanger the spillway structure or the powerplant access road. Energy dissipation downstream of the spillway is a function of both the flip bucket and plunge pool designs. The following sections discuss modifications to both these designs. Test data were recorded for equal gate openings of 3 feet, 15 feet, and fully open, representing 13,000, 53,000, and 94,000 ft^3/s discharges, respectively, at reservoir elevation 1532.0. The test plan for determining the appropriate plunge pool configuration included measuring velocities at cross sections about 150, 250, and 350 feet downstream of the spillway, measuring pressures along the plunge pool centerline, and observing flow patterns in and downstream of the plunge pool. Each plunge pool configuration was also photographed and video taped.

Original Plunge Pool Design. – The original plunge pool consisted of a 4:1 slope downstream from the powerplant access road, El. 1444.0, to the pool floor at El. 1420.0. The floor began 116 feet downstream from the spillway and was 25 feet long with a 4:1 slope up to El. 1430.0 at the end of the basin. The right side of the plunge pool was excavated on a 1.5:1 slope to El. 1450.0; the river channel formed the left side of the pool. The original plunge pool design is shown in plan on figure 9.

This pool configuration was tested with the initial 15-degree flip bucket. The energy dissipation with this flip bucket and plunge pool was inadequate. The small flip bucket deflection angle produced a flat jet impingement angle into the plunge pool. The jet penetrated the tailwater and remained in the plunge pool only for small discharges. For most discharges the jet swept through the plunge pool, confined by the river channel on the left and the hillside on the right, producing

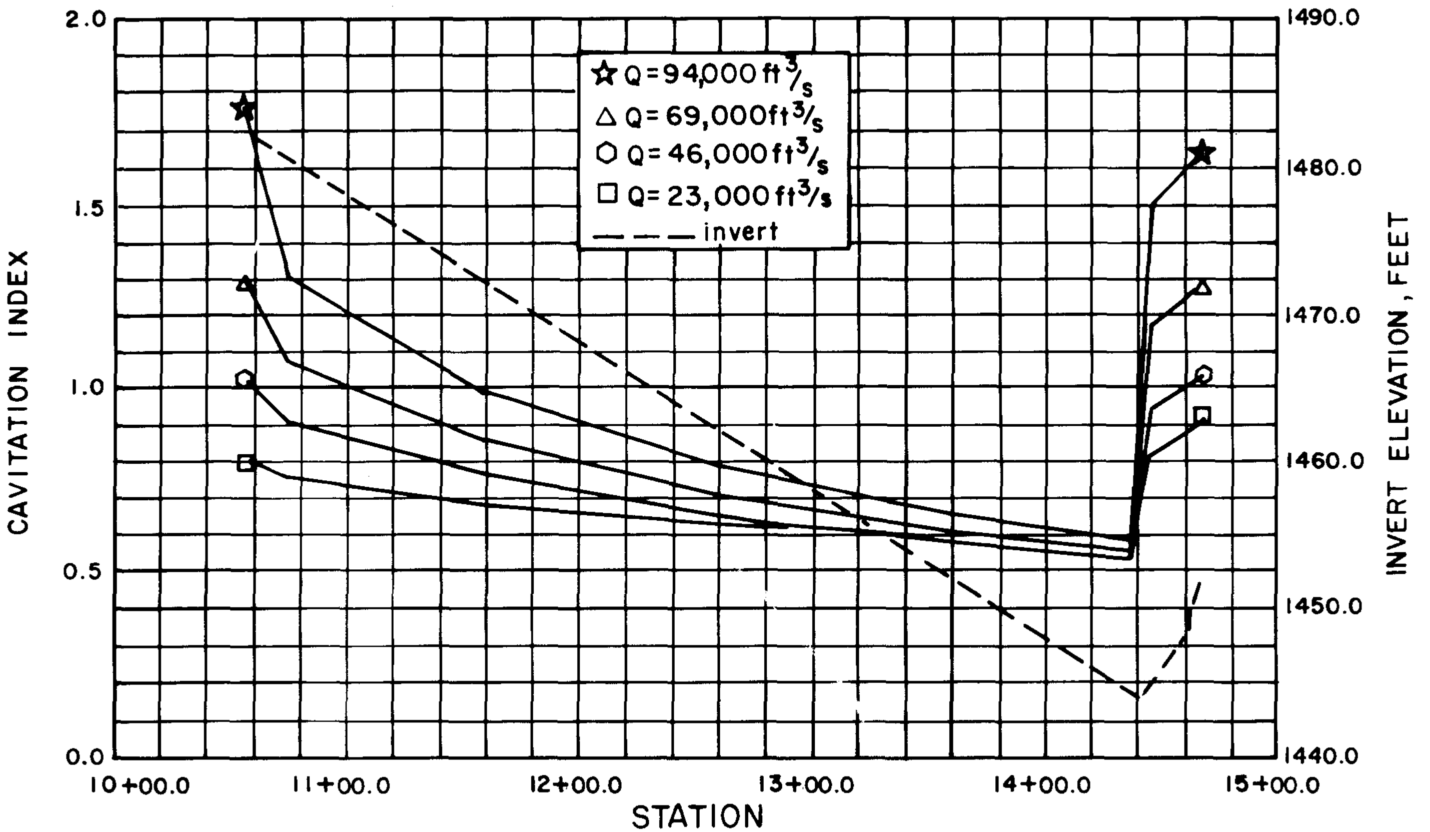


Figure 8. – Cavitation indexes for Stewart Mountain Dam auxiliary spillway.

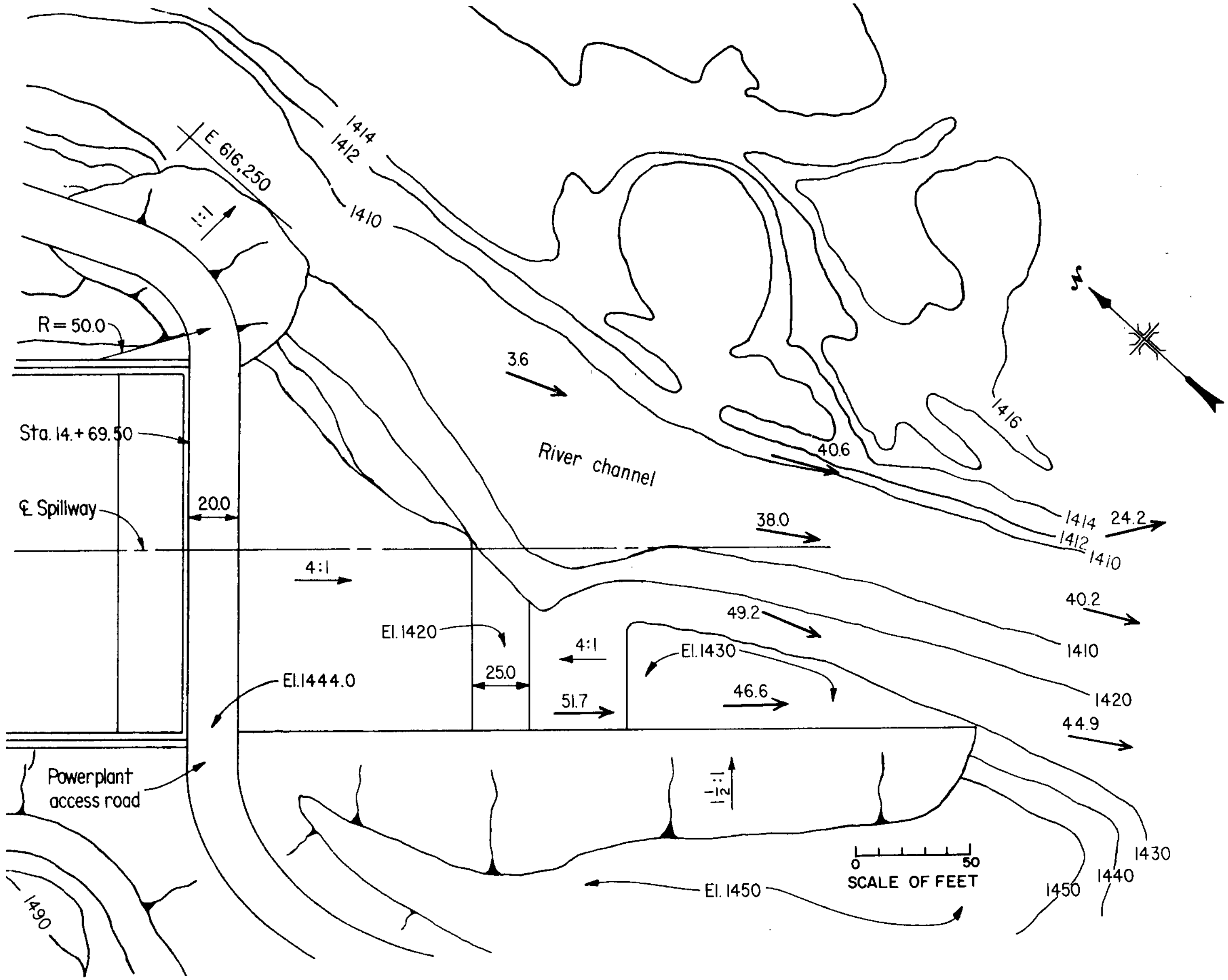


Figure 9. – Original plunge pool design. (Note: Arrows denote flow directions; numbers denote flow velocities in ft/s.)

undesirable high-velocity flow conditions along the right bank. Pressures no higher than hydrostatic were noted on the plunge pool floor for all discharges.

Flows with the gates open 3 feet (Q = 13,000 ft^3/s) impinged upon the 4:1 slope at the upstream end of the plunge pool, then entered the tailwater where a stable hydraulic jump occurred. Good energy dissipation occurred for this discharge. Generally, velocities downstream from the jump were highest along the right bank and decreased toward the river channel on the left. A backflow eddy occurred along the right bank adjacent to the hydraulic jump.

The jet with the gates open 15 feet (Q = 53,000 ft^3/s) almost entirely swept out of the plunge pool. Little energy was dissipated by an unstable jump that formed in the plunge pool. Deeper tailwater in the river channel forced the jet toward the right bank, not permitting the jet to spread. This contained the high-velocity flows near the right bank, causing flow to climb the bank. Backflow still occurred adjacent to where the jet entered the plunge pool.

The jump entirely swept out on the right side of the plunge pool during maximum discharge (Q = 94,000 ft^3/s). The jet impinged upon the pool floor then up the downstream slope and across the excavated topography at El. 1430.0. Deeper tailwater in the river channel formed a weak jump and forced the jet back toward the right bank. This concentrated the flow, producing velocities as high as 52 ft/s along the right bank (fig. 9 and table 1). No backflows occurred during sweepout of the basin. Performance of the basin for maximum discharge is shown on figure 10.

Table 1. – Velocities downstream from auxiliary spillway. Velocity ranges are given from right to left across the channel, and distances are given from downstream of the spillway. Velocities were generally higher along the right bank. Where no distance is given, the average velocity over the area is given.

Gate opening,	TW El.,	Orig. 15° bucket (fig. 9)		First mod. 15 ° bucket (fig.11)		Second mod. 15° bucket (fig. 13)		Second mod. 35° bucket
ft (Q)	ft	vel., ft/s	dist., ft	vel., ft/s	dist., ft	vel., ft/s	dist., ft	vel., ft/s
3 (13,000 ft^3/s)	1436.0	17	250	9	avg.	13	river channel	12
15	1440.0	37-15	250	24-30	250	27-17	250	
(53,000 ft^3/s)		30-7	350	32-14	350	36-20	350	mid 20's
fully open	1443.3	52	150	31-38	250	27-20	250	
(94,000 ft^3/s)						39	channel	upper 30's
		47-40	250	33-26	350	42-25	350	
		45-24	350					

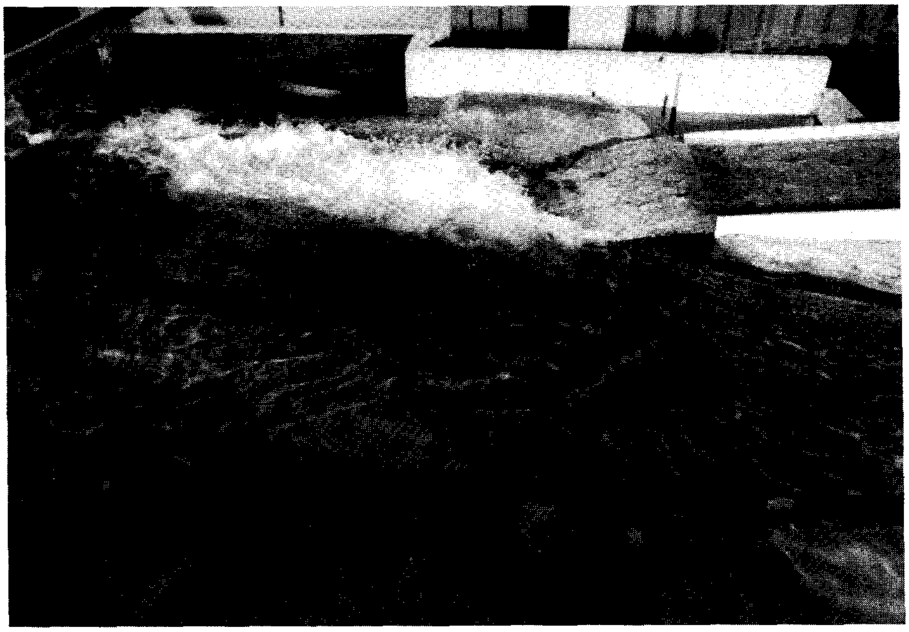


Figure 10. – Original plunge pool design operating under maximum discharge. P801-D-81059.

Little energy dissipation occurred with this flip bucket and plunge pool configuration. The flow swept through the basin, particularly on the right side, because of the small jet impingement angle and the inadequate tailwater depth. The river channel, which formed the left side of the basin, prevented the jet from spreading, producing unacceptable flow concentrations along the right bank. As a result of this poor performance, modifications were made to try to improve energy dissipation and overall flow conditions.

First Plunge Pool Modification. – The first modification lowered the plunge pool floor to produce more tailwater for energy dissipation. The pool floor was lowered 20 feet to El. 1400.0, the approximate elevation of the river channel. The upstream slope of the pool was steepened to 2.5:1 and intersected the floor 130 feet downstream from the spillway. After 25 feet, the basin sloped upward on a 0.87:1 slope to El. 1430.0. The 1.5:1 slope was maintained along the right side of the basin. The modified basin, shown in plan on figure 11, was also tested with the 15-degree flip bucket.

The velocities downstream of the plunge pool were, in general, reduced by 10 to 20 percent over the previous design (table 1). Velocities for maximum discharge, shown on figure 11, did not exceed 38 ft/s. The increased tailwater, produced by the lower floor and by the steep slope at the end of the basin, prevented entire sweepout of the hydraulic jump, even for maximum discharge. A stable hydraulic jump was maintained for a wider discharge range because of the increased tailwater. The steep slope at the right end of the floor forced a jump along the right side of the

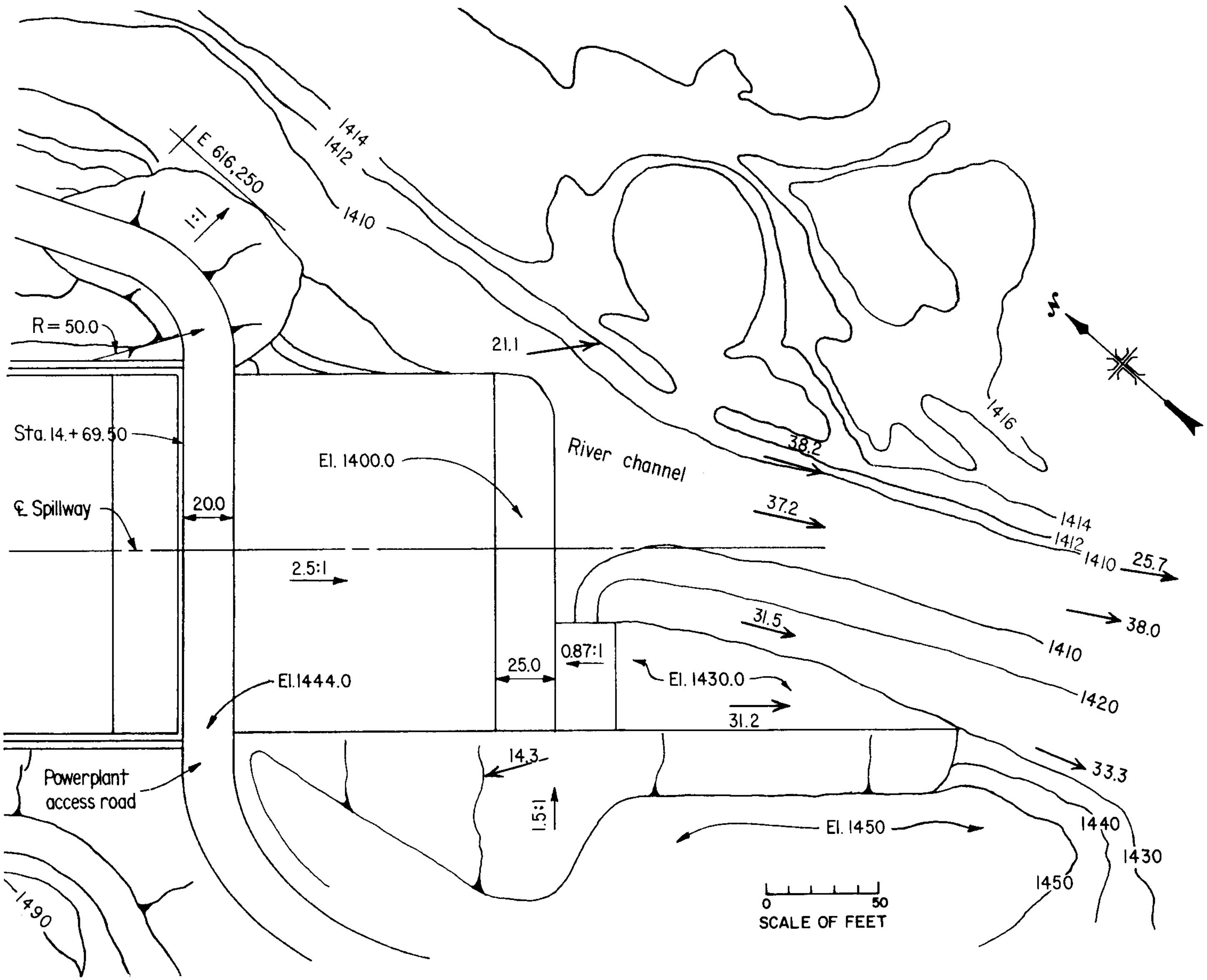


Figure 11. – First plunge pool modification. (Note: Arrows denote flow directions; numbers denote flow velocities in ft/s.)

pool by behaving similar to an end sill. Unfortunately, the impact of the high-velocity jet upon the steep slope caused much of the flow to climb the right bank. Velocities were more evenly distributed than with the initial basin. Highest velocities occurred in the middle of the river channel, rather than near the right bank. Backflows occurred along the right slope adjacent to the jet impingement area for all discharges, because the jump did not fully sweep out. No significantly high pressures were recorded along the slope upstream of or on the floor of the pool. This plunge pool is shown on figure 12 operating at maximum discharge.

Although this plunge pool configuration reduced the downstream velocities, the cost for excavating to El. 1400.0 would be prohibitive. The jet impacting heavily on the steep slope at the end of the basin created excessive flow up the right bank. The next basin modification raised the basin floor and replaced the original topography on the right side downstream from the basin.

Second Plunge Pool Modification. – The purpose of this modification was to reduce the impact of the jet on the topography downstream of the basin while retaining adequate energy dissipation. The pool floor was raised to El. 1410.0 and lengthened to 75 feet. A 1:1 slope formed the end of the basin from the floor at El. 1410.0 to the topography at El. 1430.0. This end slope was about 40 feet farther downstream than the previous basin and then blended into the natural topography along the right side of the river channel. The 2.5:1 slope at the upstream end of the basin remained. The right bank in the pool area was sloped to 1⅞:1. The modified pool, shown on figure 13, was again tested with the 15-degree flip bucket.

Velocities downstream from the plunge pool were as high as those of the original design, but distributed differently (table 1). A hydraulic jump formed for low discharges, but as the discharge increased, the ability of the plunge pool to dissipate energy decreased. The jet continued to impinge heavily on the slope downstream of the basin. A large amount of flow rose over the right bank with the remainder shooting toward the river channel. Flow entering the left side of the pool followed the river channel. Backflows continued to occur adjacent to the hydraulic jump on the right side of the basin.

This plunge pool was also tested with a vertical wall along the right side instead of the 1⅞:1 slope. Backflows disappeared with the addition to the vertical wall. Flow conditions were more stable with the vertical sidewall; however, the jet still swept across the pool floor producing no significant pressures and little energy dissipation. This plunge pool is shown operating under maximum discharge with the vertical right sidewall on figure 14.

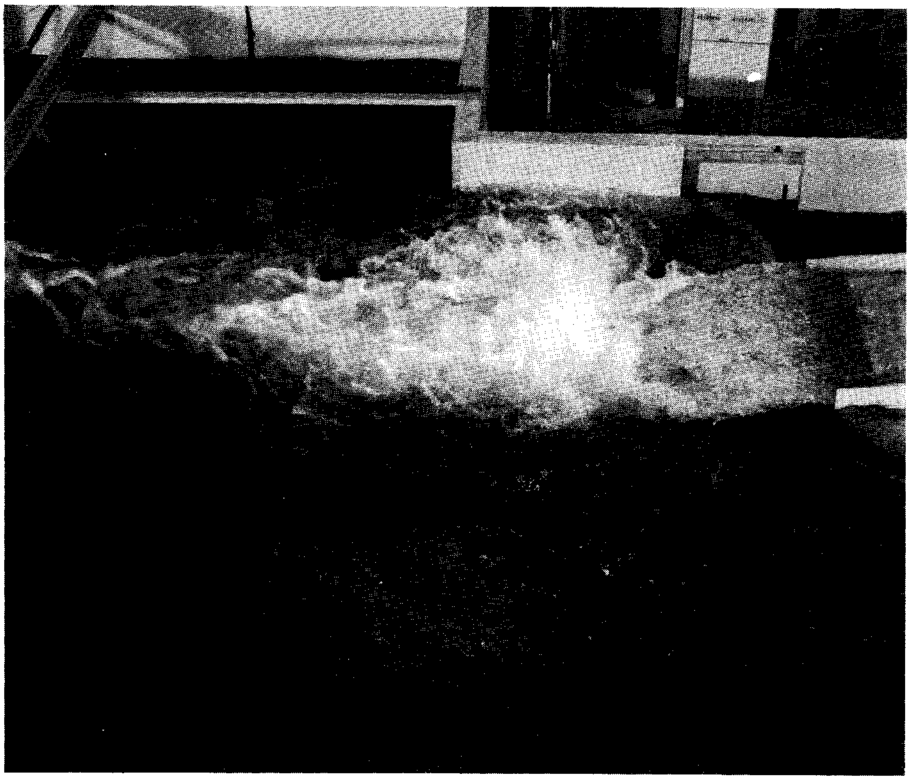


Figure 12. – First plunge pool modification operating under maximum discharge. P801-D81060.

Thirty-Five-Degree Flip Bucket Tested with the Second Plunge Pool Modification. – Previous plunge pool designs were tested with the 15-degree flip bucket. During these tests the jet swept through the plunge pool during high discharges. Little energy dissipation occurred as a result of the plunge pool design and the flat impingement angle of the jet. Therefore, the flip angle was increased to 35 degrees above horizontal for the following tests. The flip bucket P.C. (point of curvature) was at Sta. 14+37 and El. 1444.3. The invert was formed by a 45-foot radius ending at Sta. 14+67.29 and El. 1452.22. This flip bucket design is shown on figure 4. The plunge pool design from the previous tests was used.

The steeper jet impingement angle into the plunge pool greatly improved energy dissipation. The 35-degree flip bucket angle threw the jet farther downstream, protecting the area immediately downstream of the spillway from possible erosion damage. Impact pressures on the basin floor were increased, but not significantly. Velocities downstream of the plunge pool were only slightly reduced. A discharge of 13,000 ft^3/s from the 35-degree flip bucket is shown on figure 15.

The velocities recorded for each modification to either the plunge pool or flip bucket are shown in table 1. These velocities were taken with an initial tailwater El. of 1434.8 produced by a discharge

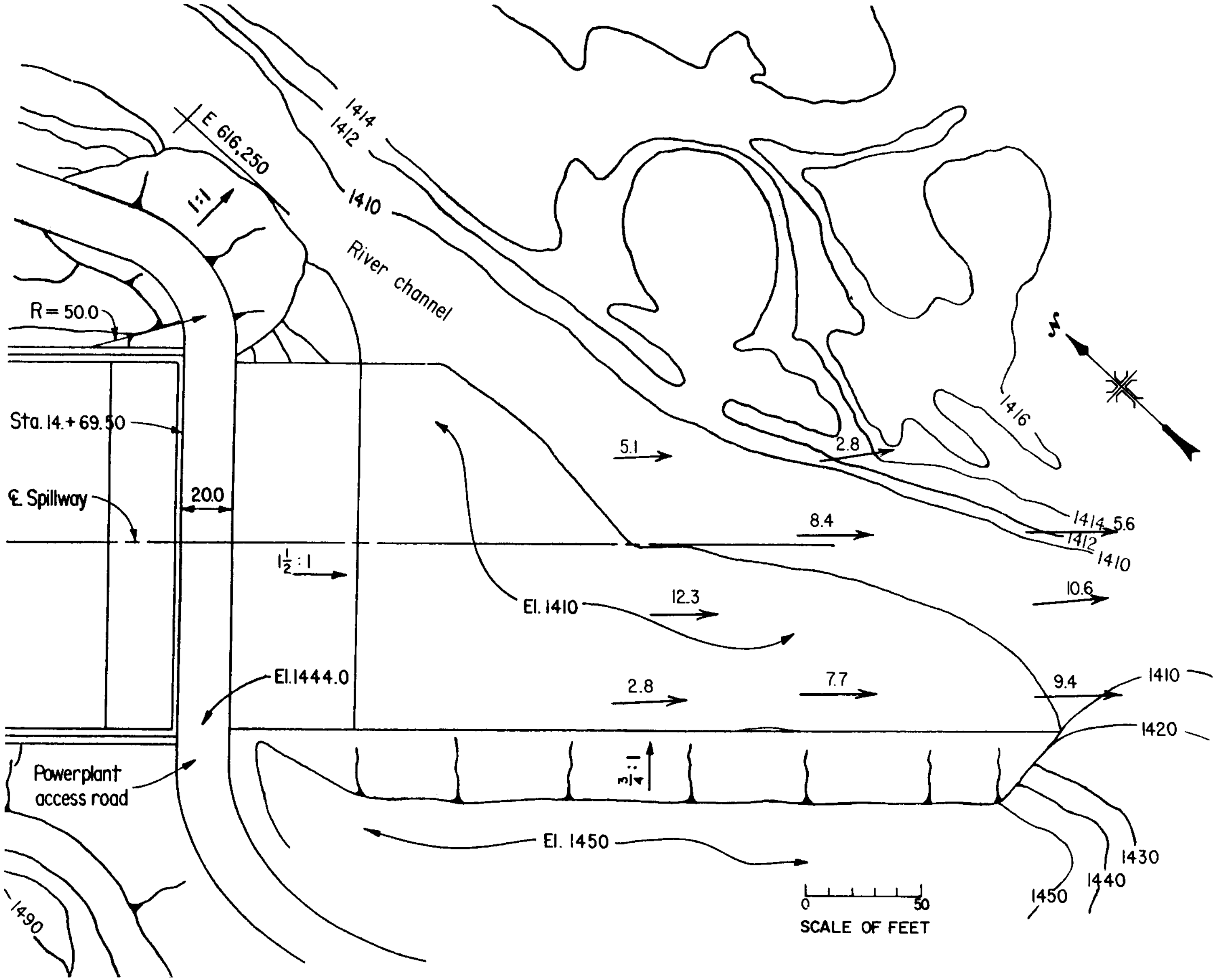


Figure 13. – Second plunge pool modification. (Note: Arrows denote flow directions; numbers denote velocities in ft/s.)

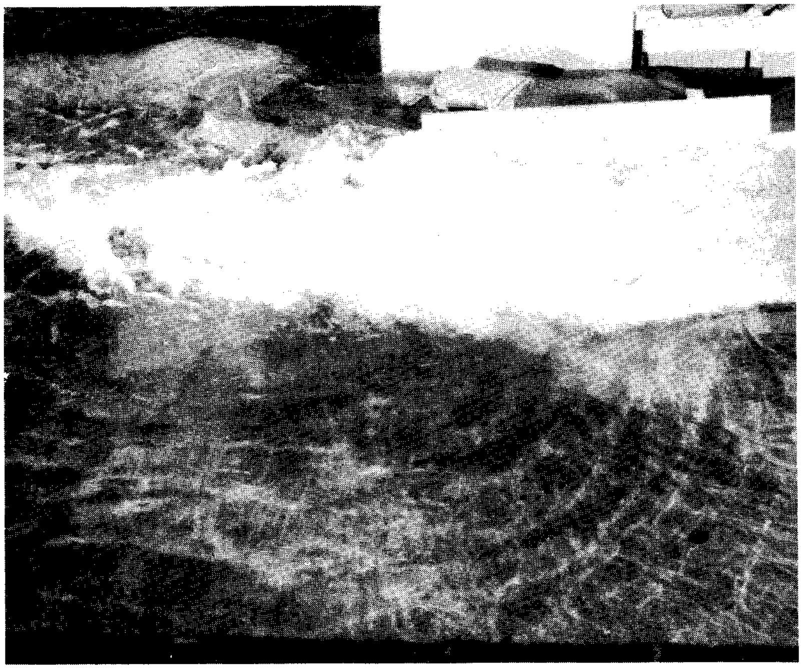


Figure 14. – Second plunge pool modification operating under maximum discharge. P801-8-81061.

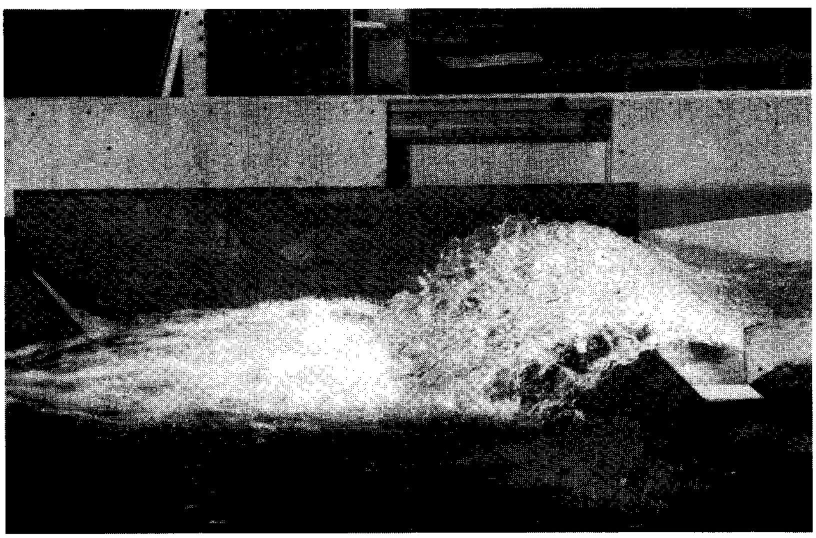


Figure 15. – Second plunge pool modification and recommended flip bucket for $Q = 13{,}000\ ft^3/s$.

of 120,000 ft^3/s from the existing spillway. Velocities at each measurement location for maximum discharge and each modification are shown on figures 9, 11, and 13.

Recommended Plunge Pool and Flip Bucket Design. – The following recommended plunge pool and flip bucket designs were tested with the final operating criteria. These criteria were 75,000 ft^3/s passed by the existing spillway (TW El. 1430.0), adding 94,000 ft^3/s through the auxiliary spillway (TW El. 1439.4), then increasing the existing spillway to 120,000 ft^3/s, for a combined total discharge of 214,000 ft^3/s.

The 35-degree flip bucket is the recommended design. This flip bucket provided two major advantages:

- A steeper impingement angle producing less tendency for the jet to sweep out of the plunge pool and greater energy dissipation.

- The jet impact area was farther downstream, protecting the area immediately downstream of the spillway from possible erosion damage.

Improved flow conditions were observed with operation of the 35-degree bucket and the second plunge pool design. However, the plunge pool design needed some modification to maximize the benefit of the 35-degree flip bucket. Use of the 35-degree flip bucket with the following plunge pool configuration provided the best possible flow conditions.

The major final plunge pool modification consisted of removing the hillside that protruded into the right side of the pool. The hillside had been used in previous plunge pool configurations to help force a hydraulic jump; however, this approach was abandoned because a stable jump would not form and impingement of the jet on the hillside caused excessive flow over the right bank. The following modifications were adopted for the final design:

- Extend the pool floor at El. 1410.0 to about 250 feet downstream.

- Excavate the entire right side of the pool on a 0.75:1 slope from the pool floor at El. 1410.0 to El. 1450.0. Continue this excavation about 250 feet downstream to where the slope meets the original topography.

- Steepen the upstream slope between the powerplant road (El. 1444.0) and the pool floor to a 1.5:1 slope. The recommended plunge pool is shown on figures 6 and 16.

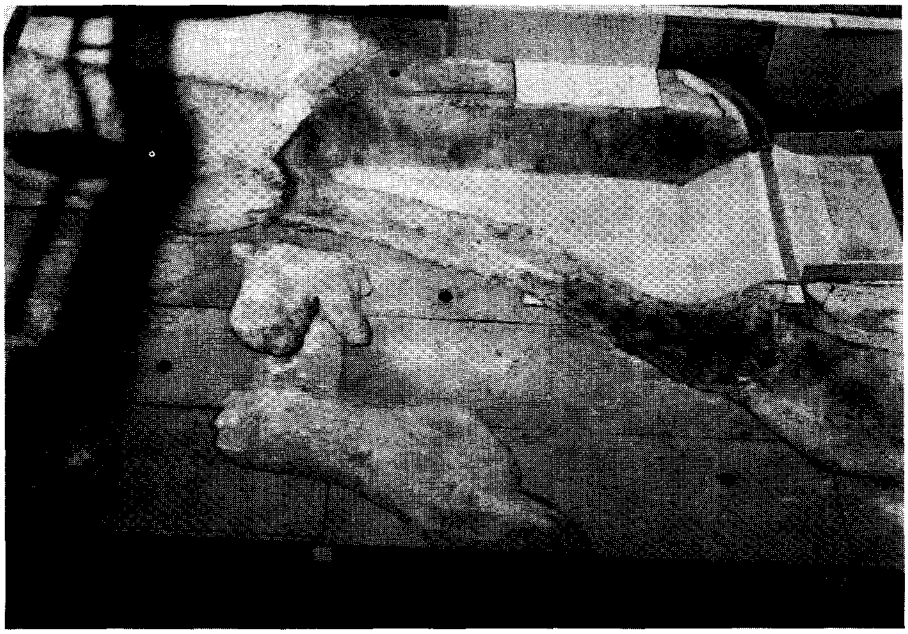


Figure 16. – Recommended plunge pool. P801-D-81062.

The steeper flip bucket angle and upstream slope prevented impingement on the slope for all discharges after the jet swept out of the spillway chute. The hydraulic jump in the chute during initial gate opening caused flow over the end of the bucket. The flow impinged on the powerplant road and flowed down the slope into the plunge pool (fig. 17). This flow condition will require protection for the road and possibly the slope, depending on the condition of the excavated rock.

Removing the protruding hillside at the end of the pool on the right side greatly improved flow conditions. A uniform hydraulic jump formed for low to medium range flows. Uniformly distributed, less turbulent flow occurred during maximum discharge. The river channel forming the left side of the pool continued to partially restrict the spread of the jet, but did not force the flow toward the right bank as had previously occurred. No backflows occurred along the right bank for any discharge.

The steeper slope upstream of the basin was particularly beneficial during the low discharge range. The jet from the 3-foot gate openings entered the basin at the base of 1.5:1 slope, where previously the jet had impinged upon the excavated rock before entering the tailwater. The steeper slope combined with the greater flip bucket angle allowed the jet to spring entirely over this area and into the tailwater to form a jump. The velocity distribution for Q = 13,000 ft³/s is shown on figure 18. Maximum velocities downstream of the jump were 12.3 ft/s at 150 feet downstream, 8.4 ft/s at 250 feet downstream, and 10.5 ft/s at 350 feet downstream. The maximum pressure, equivalent to 30 feet of water, was measured at the intersection of the upstream slope and the

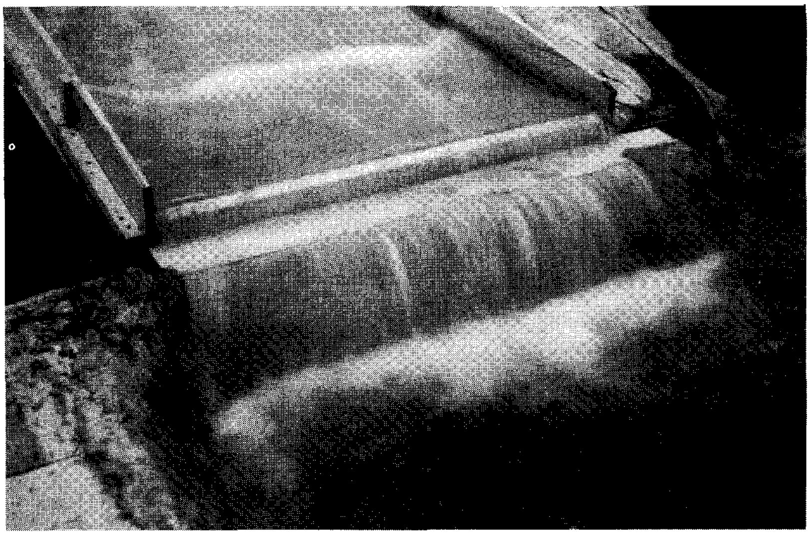


Figure 17. – Hydraulic jump in the spillway chute and flow onto the powerplant road. P801-D-81063.

pool floor. This recommended pool and flip bucket design provided excellent energy dissipation for this discharge (fig. 19).

The jet produced by 15-foot gate openings entered the plunge pool about 130 feet downstream from the flip bucket. A hydraulic jump still formed, but was close to sweepout. Some energy dissipation occurred. Maximum velocities downstream of the jump were 22.5 ft/s at 250 feet and 24.5 ft/s at 350 feet downstream of the spillway. The velocity distribution is shown on figure 20. This distribution confirmed observations of a uniform flow distribution downstream of the plunge pool (fig. 21). The maximum pressure of 35 feet was measured about 164 feet downstream from the flip bucket.

Flow conditions in and downstream of the plunge pool under maximum discharge, 94,000 ft^3/s, were improved over previous plunge pool configurations (fig.22). The jet entered the tailwater about 130 feet downstream of the flip bucket. The jet then impinged and swept across the floor of the basin about 50 feet before forming a weak hydraulic jump. Energy dissipation from the jump, however, was minimal. Velocities downstream were not reduced, but were more evenly distributed than those of previous plunge pool designs. Maximum velocities downstream of the pool were 34.4 ft/s at 150 feet, 29.9 ft/s at 275 feet, and 39.9 ft/s at 400 feet downstream (fig. 23). Flow across the entire width of the pool was uniform. Waves, about 300 feet downstream from the spillway, caused slight overtopping of the right bank at El. 1450.0.

Maximum pressures were produced by maximum discharge and were measured on the pool floor about 165 feet downstream from the flip bucket. With the tailwater produced by the final operating

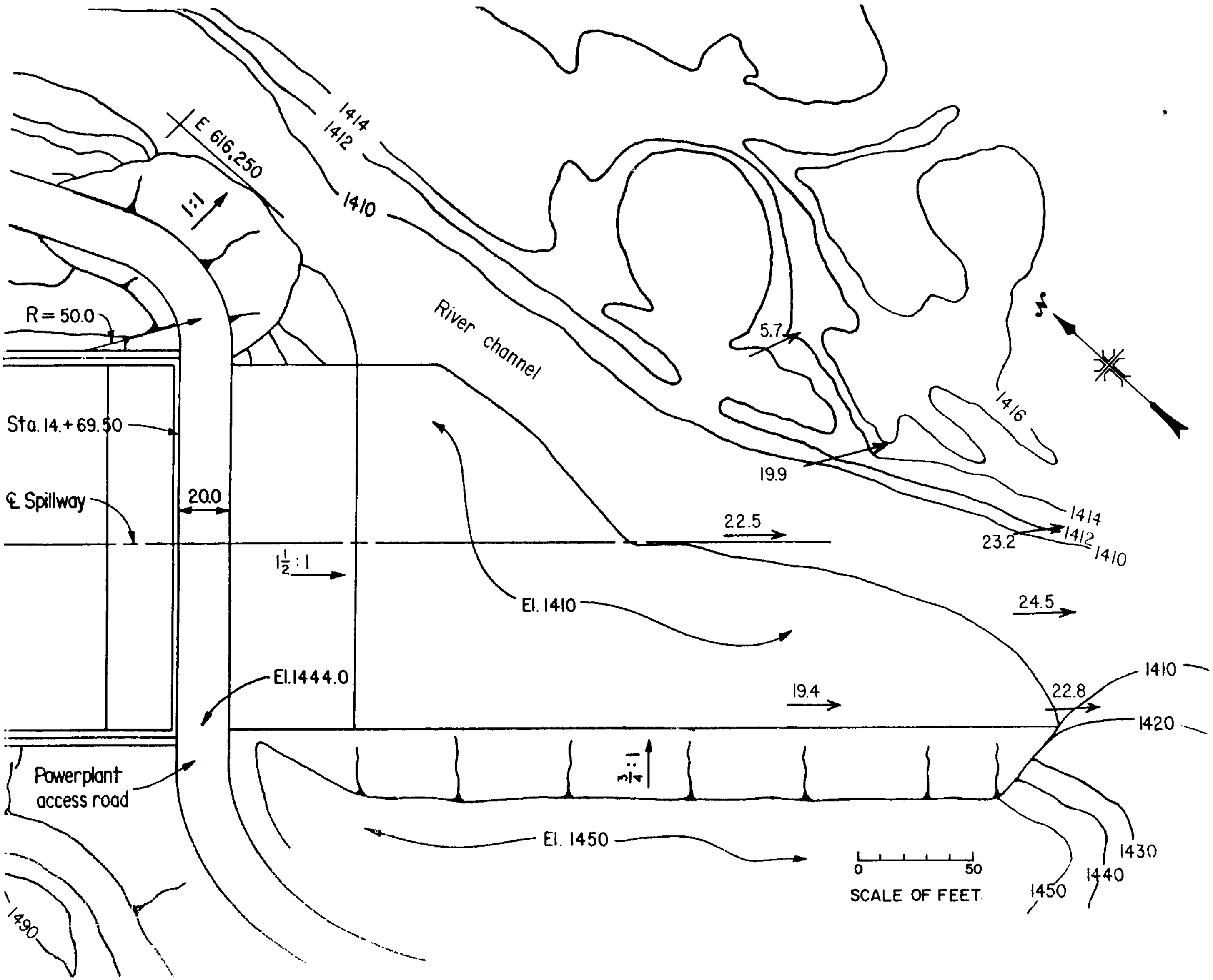


Figure 18. – Velocity distribution for Q = 13,000 ft³/s, recommended plunge pool design. (Note: Arrows denote flow directions; numbers denote velocities in ft/s.)

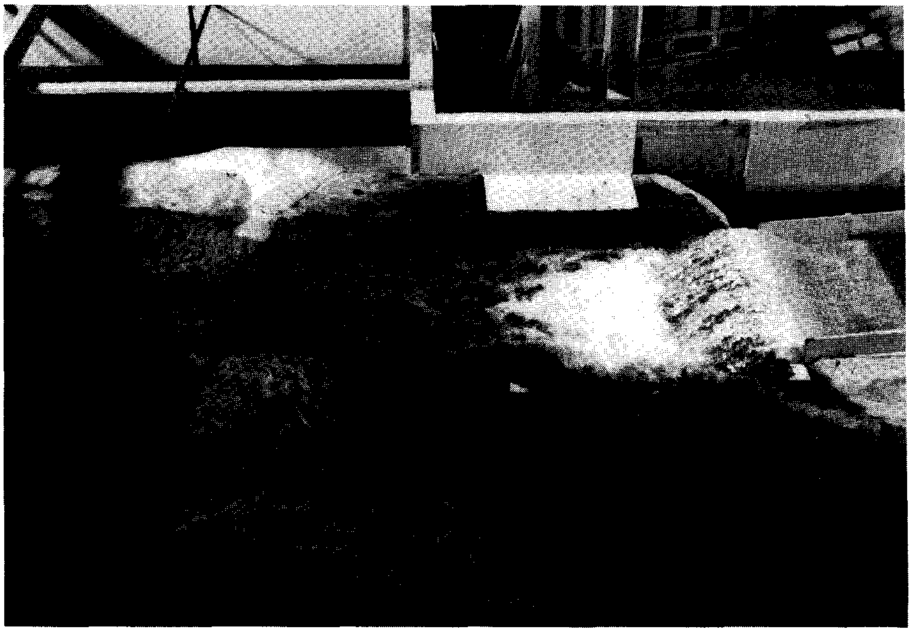


Figure 19. – Recommended plunge pool design, Q = 13,000 ft³/s. P801-D-81064.

criteria, the maximum pressure was 49 feet, approximately 19.6 feet above the tailwater. The maximum pressure of 54.4 feet was recorded when testing was done assuming the worst case of no flow from the existing spillway before auxiliary spillway operation. Even this condition did not produce excessive static or fluctuating pressures.

An erodible bed downstream of the auxiliary spillway was not modeled because only qualitative information could have been obtained. It was determined that flow patterns observed from the fixed-bed model would produce as much information as an erodible-bed model. The jet will eventually erode a plunge pool with the depth dependent upon the amount and length of discharge and the integrity of the rock. The river channel along the left side of the plunge pool slightly restricted the spread of the jet. However, as erosion occurs this effect will diminish. The uniform cut slope along the right bank should produce more predictable and less damaging erosion. Erosion damage below the spillway should not produce any excessive problems based upon the geologic data from the site, which indicates that the rock in the plunge pool area is not fractured and has good strength.

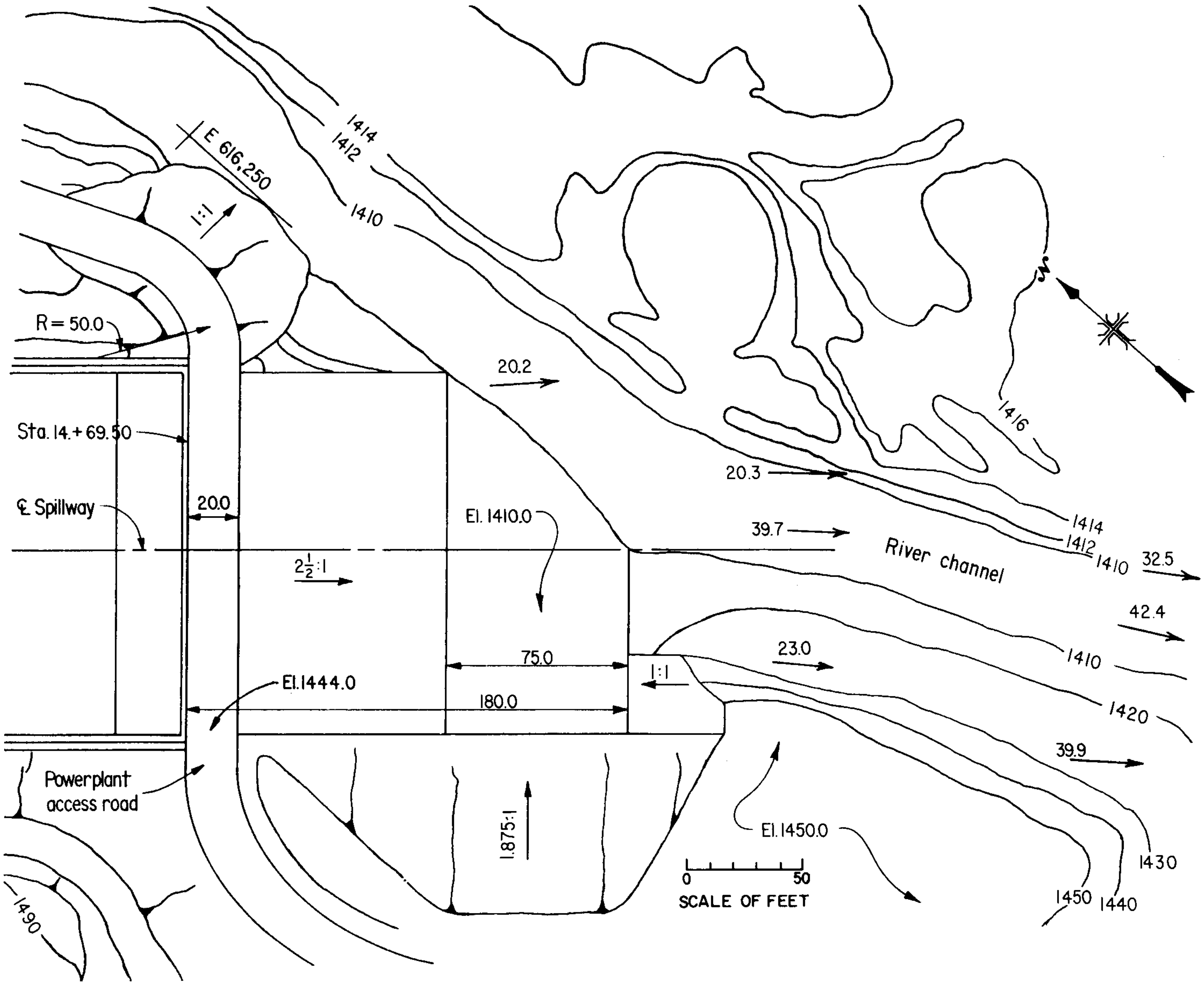


Figure 20. – Velocity distribution for Q = 53,000 ft³/s, recommended plunge pool design. (Note: Arrows denote flow directions; numbers denote flow velocities in ft/s.)

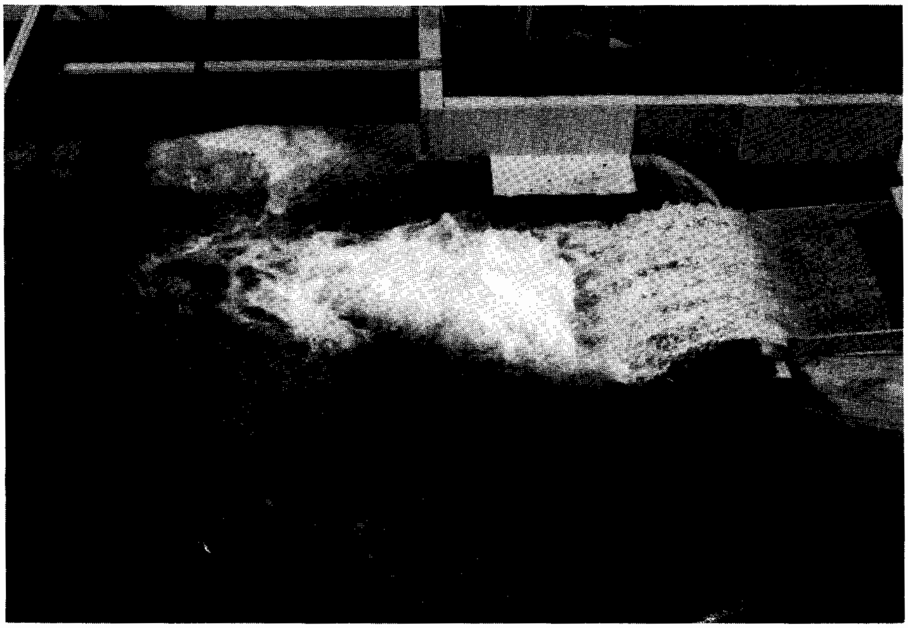


Figure 21. – Recommended plunge pool design, Q = 53,000 ft³/s. P801-D-81065.

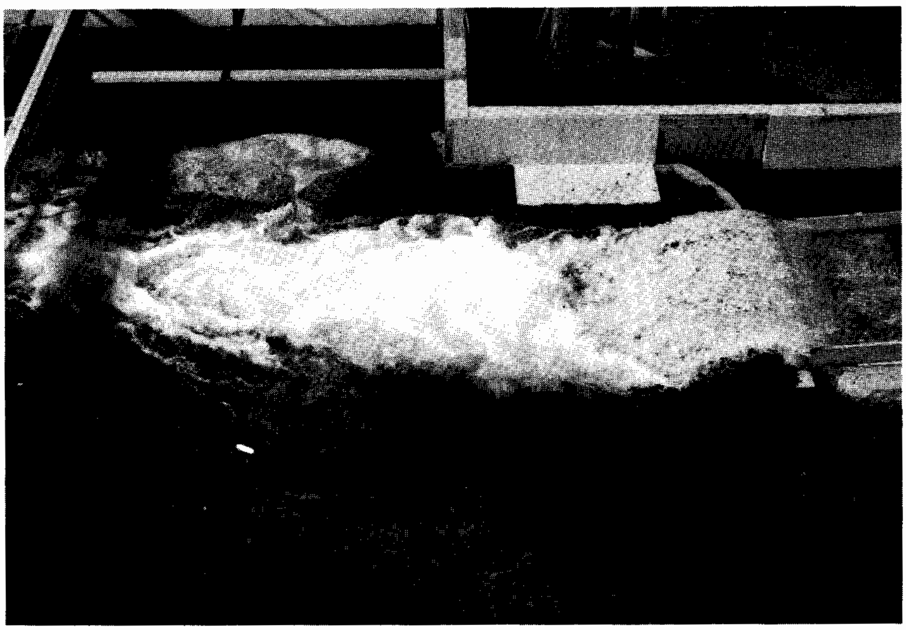


Figure 22. – Recommended plunge pool design, Q = 94,000 ft³/s. P801-D-81066.

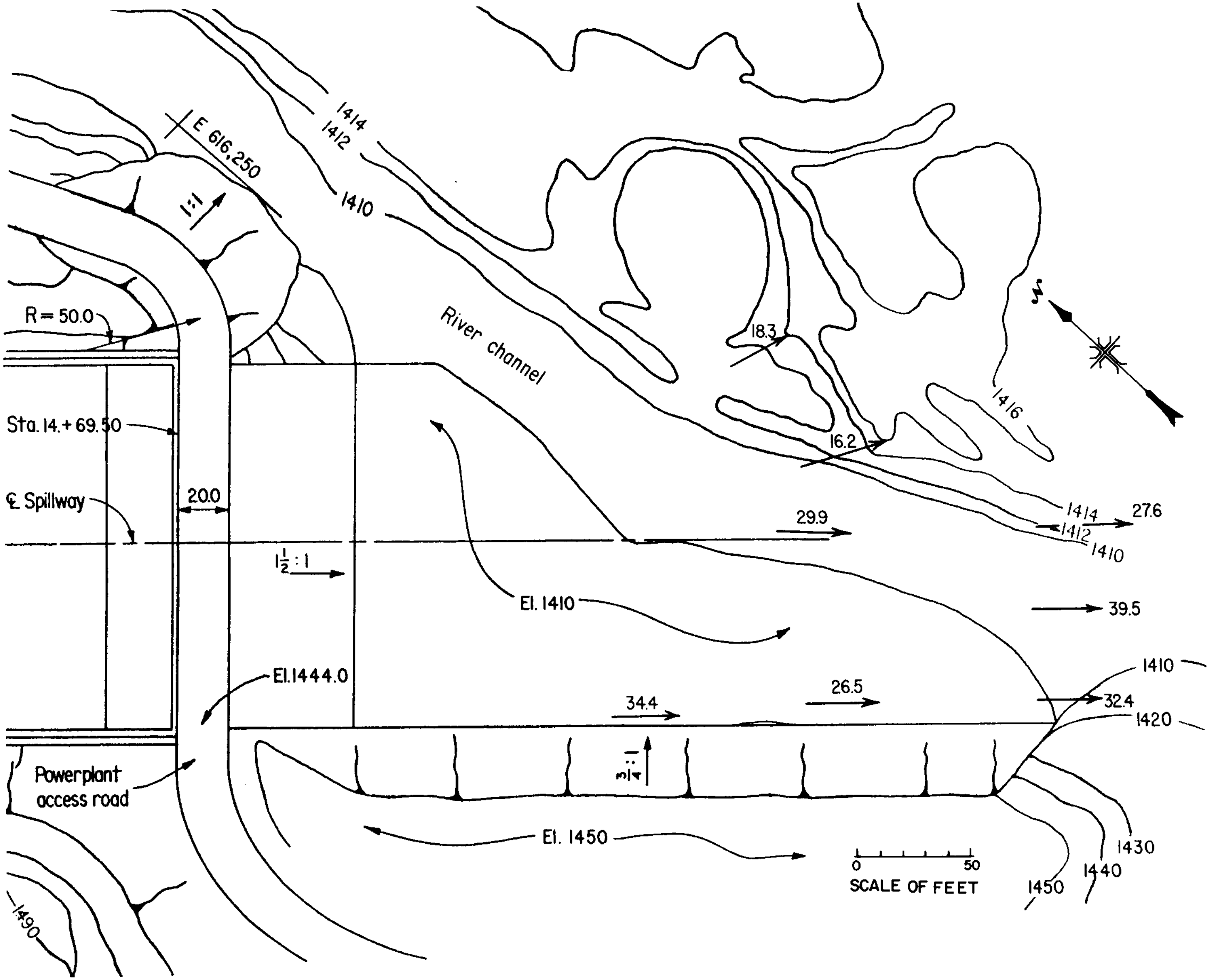


Figure 23. – Velocity distribution for Q = 94,000 ft³/s, recommended plunge pool design. (Note: Arrows denote flow directions; numbers denote flow velocities in ft/s.)

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.