HYDRAULIC MODEL STUDY OF STEWART MOUNTAIN DAM RIGHT ABUTMENT AUXILIARY SPILLWAY

BY

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Memorandum
Chief, Concrete Dams Branch

Denver, Colorado
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Chief, Hydraulics Branch

Stewart Mountain Dam Hydraulic Model Study

Attached is a report of the hydraulic model study performed for the right abutment auxiliary spillway at Stewart Mountain Dam. The report includes a summary of the model study results.

A draft GR series report will be available for the Stewart Mountain design team and consultants meeting in late August.

GPO 852320

Philip H. Burgi

Attachment

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HYDRAULIC MODEL STUDY
OF
STEWART MOUNTAIN DAM
RIGHT ABUTMENT AUXILIARY SPILLWAY

by

K. L. Houston

INTRODUCTION

An evaluation of Stewart Mountain Dam was completed under the Safety of Dams Program. As a result, a need for greater spillway capacity was identified and designs developed for addition of an auxiliary spillway on the right abutment of the dam. This report will summarize the hydraulic model studies conducted on the proposed right abutment spillway for the existing dam at Stewart Mountain.

PURPOSE

The model study was undertaken 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 (probable maximum flood). The initial auxiliary spillway design is shown in plan and section on figure 1. The model investigation included the following features:

- Spillway approach channel configuration and flow velocity distribution patterns

- Spillway discharge capacity, chute water surface profiles, and unequal gate operation

- Characteristics of the spillway flip bucket

- Potential for erosion downstream of the spillway, determined by velocity and pressure measurements

The result of these studies will be summarized in the following paragraphs.

OPERATING CRITERIA

The existing spillway, on the left abutment, will be used as the service spillway. The resulting tailwater will affect energy dissipation of discharges from the auxiliary spillway. Operating criteria initially required the existing spillway to pass 120,000 ft^3/s (tailwater El. 1434.8) before operating the auxiliary spillway. Most model testing was completed under these criteria. Near the end of the test program the operating criteria were changed, requiring the existing spillway to pass 75,000 ft^3/s (tailwater El. 1430), then adding 94,000 ft^3/s (tailwater El. 1439.4) from the auxiliary spillway before increasing the existing spillway flow to 120,000 ft^3/s producing a combined spillway discharge of 214,000 ft^3/s at tailwater El. 1443.3. The recommended auxiliary spillway plunge pool basin was evaluated under the tailwater produced by these latter operating criteria.

THE HYDRAULIC MODEL

The 1 to 40 scale model included: 600 ft of the reservoir and spillway approach, the 150-ft wide auxiliary spillway controlled by four 30-ft by 33-ft radial gates, the spillway chute and flip bucket, and about 600 ft of river channel downstream of the spillway. The model was sized to allow investigation of flow patterns across the entire width of the 600-ft-long downstream channel. Topography modeled for the river channel included an area about 320 ft to the left of the channel, and the hillside to the right of the channel from the river level to El. 1450. Operation of the model showed the area modeled was adequate because all excessive wave heights or flow velocities dissipated within the modeled area.

The existing spillway on the left abutment was not modeled. Prior to constructing the model it was determined that correct energy dissipation below the auxiliary spillway would be modeled without supplying discharges from the left spillway, provided that correct tailwater depths were modeled.

An overall view of the 1 to 40 scale model is shown on figure 2.

SUMMARY

The following recommendations are based upon model study results:

Approach Channel

Initial approach channel geometry and recommended changes are shown on figure 3.

A vertical semicircular guidewall was placed adjacent to the left side of the spillway entrance. This guidewall, which extended from the left pier into the reservoir, improved the direction of the flow into the spillway, thus increasing spillway discharge.

A vertical wall should be placed between the right side of the spillway entrance and the embankment slopes which form the right side of the approach channel.

The model did not include the area where the existing reservoir stilling well is located. However, this location (on the face of the dam near the right thrust block) should be sufficiently removed from the effects of drawdown produced by operation of the right spillway.

Discharge Capacity

At maximum reservoir El. 1532, a maximum discharge of 94,000 ft^3/s is attained 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-ft gate opening increments (figure 4).

Gate Operation

Uniform gate operation is recommended. Approach channel geometry produces an uneven flow distribution in the channel, thus affecting uniformity of flow through the gates. The discharge curves were developed for uniform gate operation. With all gates equally open the discharge will be accurately predicted by the discharge curves; opening one or two gates alone will allow only an approximation of the flow (e. g., at reservoir El. 1532, 3-ft uniform operation = 13,000 ft^3/s, one gate open 12 ft will not necessarily produce 13,000 ft^3/s).

Best possible flow conditions for nonuniform operation at maximum reservoir operation are produced by opening the far right gate (looking downstream) first. This gate may be opened alone until reaching an 18-ft gate opening. To further increase the discharge, the far left gate may be opened to 18 ft yielding a total discharge of approximately 32,000 ft^3/s. Releases above this should be accomplished by opening the two center gates as evenly as possible to equal the two outside gate openings.

Flip Bucket

Two flip buckets were tested: the initial 15° (above horizontal) bucket and the recommended 35° bucket. The 35° bucket, formed by a 45-ft radius beginning at Sta. 14+37, provided more uniform flow conditions in the plunge pool and the downstream river channel.

With initial opening of the gates a hydraulic jump forms in the chute upstream of the flip bucket. For the 35° bucket the jump will sweepout when the discharge reaches about 6,000 ft^3/s. Until sweepout occurs, flow over the end of the flip bucket will impinge upon the powerplant roadway.

Chute Wall Heights

The chute side walls must contain flow depths associated with the maximum discharge and the hydraulic jump upstream of the 35° flip bucket prior to sweepout. Flow depths were measured along the chute wall normal to the slope. Flow depths for maximum discharge, were:

Location	Depth
Sta. 10+42	24.5 feet
Sta. 11+75.5	13.5 feet
Sta. 14+05	12.9 feet
Sta. 14+69.50	17.5 feet

Sweepout of the hydraulic jump occurs at about 6,000 ft^3/s. Prior to 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 slope upward to contain a flow depth of 13.5 ft at Sta. 14+05 to 20 ft at Sta. 14+45. The flow depth associated with the jump is greater than the flow depth at maximum discharge and therefore governs the wall heights at the flip bucket.

Plunge Pool Excavation

The upstream boundary of the plunge pool, from the powerplant road (El. 1444) to the pool floor (El. 1410), should be cut to a 1-1/2 to 1 slope. The right bank of the plunge pool should be cut on a 3/4 to 1 slope from the floor to El. 1450, continuing downstream for about 250 ft where the cut slope will meet the existing topography. The left side of the plunge pool is outlined by the river channel. This pool configuration, figures 5 and 6, provided the best flow distribution in the river channel downstream. Velocities downstream of the basin for maximum discharge were as evenly distributed as possible, but consistently in the upper 30 ft/s range.

The proposed location for the helicopter pad on the hillside adjacent to the basin on the right side will experience a great deal of spray during medium to high range releases from the auxiliary spillway and occasional waves during maximum discharge.

INVESTIGATION

Spillway Approach Channel

Proper alignment of the auxiliary spillway with the river channel will require extensive excavation in the reservoir for an approach channel. The 150-ft wide approach channel will be excavated to El. 1486 on a 250-ft radius which produces about a 75° turn from the reservoir to the spillway entrance. The cut slope from the floor of the channel along the right side will be 3/4 to 1 to a 20-ft-wide berm at El. 1530. The cut slopes forming the right side of the channel above El. 1530 will be 1-1/4 to 1 with 10-ft-wide berms every 30 ft in elevation. The left side of the channel will also be excavated. The initial design included a vertical, 50-ft radius quarter round section from the left pier into the reservoir topping out at El. 1500. This will be extended further into the reservoir by a 3/4 to 1 slope from El. 1500 to the bottom of the channel (figure 3, original design).

Approach channel geometry was investigated because the change in flow direction upstream of the spillway crest could significantly affect the spillway discharge capacity. Initial model operation revealed high velocity flow along the face of the dam and over the topography at El. 1500. This flow, perpendicular to the spillway centerline, produced a large contraction around the left pier and significantly reduced discharge through the left spillway bay.

To increase flow through the left spillway bay several arrangements for extending the left pier or providing a guidewall extending into the reservoir were investigated. The optimum solution was increasing the height of the original guidewall formed by a 50-ft radius to El. 1532, then extending the wall to the left along the face of the dam for about another 50 ft, almost forming a semicircle (figure 3). This significantly improved flow through the left bay and increased the maximum spillway discharge by about 4,000 ft^3/s.

On the right side near the dam, the approach channel geometry abruptly changed from the 3/4 to 1 slope to the vertical face of the dam and spillway end pier. The spillway end pier also partially extended into the reservoir. This geometry produced a slight contraction around the end pier during higher discharges.

The right side of the approach channel was modified to alleviate this contraction by installing a warped surface from the 3/4 to 1 cut slope to the vertical pier. Because the warped surface may be expensive to construct, the final recommendation was to construct a vertical wall from the upstream end of the pier nose to the excavated rock slope.

Velocities were measured at four stations in the approach channel (figure 3). Measurements were recorded at the base of the 3/4 to 1 slope, the centerline of the channel, and about 10 ft away from the left guidewall at 0.2, 0.5, and 0.8 of the reservoir depth. Velocities increased from right to left across the channel and as the flow approached the spillway. Velocities were generally low except by the left guidewall. Average velocities 10 ft from the left guidewall for three discharges at reservoir El. 1532 were as follows:

Discharge (ft^3/s)	Velocity (ft/s) Station 4	Velocity (ft/s) Station 1
13,000	1.2	2.5
53,000	4.5	11.3
94,00	8.4	22.6

A concrete guidewall is recommended because of the high velocities at maximum discharge.

Spillway Discharge Capacity

The 150-ft-wide spillway is controlled by four 30- by 33-ft radial gates atop a low ogee crest followed by a 10 to 1 sloping chute and a flip bucket terminal structure. Discharge rating curves were developed for this gate controlled crest. Free flow spillway discharge was 94,000 ft^3/s at maximum reservoir elevation. Discharge curves were developed with all four gates opened equally in 3-ft increments up to a 24-ft opening (figure 4).

Spillway discharge capacity was affected by the approach channel geometry. The primary effect was the flow phenomena created by the channel bend. The bend produced superelevation of the flow surface, a rise along the outer bank (right side) and a decrease near the inside of the bend or left side of the spillway. This was indicated by observation of the flow and velocity measurements. The effect of the bend was also observed by the direction of the fins downstream of the spillway piers. The fin downstream from the left pier was directed substantially toward the right, with this effect dissipating across the chute toward the right. The fins downstream from the piers will produce significant spray in the prototype, especially for intermediate discharges.

Gate Operation

Uniform gate operation is recommended. Approach channel geometry produces uneven flow distribution upstream of the spillway gates as was discussed in the previous section. Because of this, nonuniform gate operation may produce inaccuracies in the discharge when predicting flow from the discharge curves which were developed for uniform operation.

The best possible flow condition, with unequal operation, is produced by opening the far right gate (looking downstream) first. This gate may be opened alone to an 18-ft opening and sweepout the jump as soon as possible. Increasing above this discharge to approximately 32,000 ft^3/s, should be done by next opening the far left gate to 18 ft. Releases greater than this amount should be accomplished by opening the center two gates symmetrically to equal the two outside openings.

Unequal opening of any gate above 18 ft (about 32,000 ft^3/s) will produce undesirable flow conditions in either the chute or plunge pool. Gates unequally open above 18 ft do not allow a smooth transition from unequal to equal operation.

Operation of the two right gates or the two left gates simultaneously should be avoided because vortices will form in the operating bays.

Nonuniform operation, not following the above procedure, may cause overtopping of the chute walls and excessive spray from rooster tails or fins. Either of the conditions could cause water to flow down the outside of the spillway chute and subsequent erosion damage.

Chute Water Surface Profile

The water surface profile along the wall, normal to the chute slope, was measured for maximum discharge. This profile indicated a maximum flow depth of 24.5 ft at Sta. 10+42 decreasing to 13.5 ft at Sta. 11+57.5 then to 12.9 ft at the beginning of the flip bucket. During initial opening of the spillway gates, a jump forms in the chute upstream of the 35° flip bucket. The jump will remain in the chute until the discharge reaches about 6,000 ft^3/s, causing sweepout. The water surface profile of the jump, not the flow depth at maximum discharge, will govern the wall height in the area of the flip bucket. The walls should be high enough to contain a 13.5-ft flow depth at Sta. 14+05 and slope up to contain a 20-ft depth at Sta. 14+45. The flow depth of 17.5 ft at the end of the spillway (Sta. 14+69.5) for maximum discharge will be contained by the wall height necessary for the hydraulic jump. These flow depths should allow a reduction in the proposed 20-ft wall height along the majority of the chute; however, because of the recommended 35° flip bucket design, wall height near the flip bucket should be increased.

Velocities and cavitation potential were computed for the entire length of the chute and flip bucket. Maximum discharge and flow rates of one-fourth, one-half, and three-fourths of maximum discharge were analyzed. All values of the cavitation index or flow sigma were greater than 0.2; therefore, cavitation should not occur along the chute or on the flip bucket. Because the cavitation potential is low, the ends of the chute underdrains and the openings for the flip bucket drains may be left uncovered. The flip bucket drains will not require eyebrows provided that the ratio of the vertical depth of the drain opening to the drain diameter is greater than or equal to one.

Potential for River Channel Erosion

Historically, erosion damage has been a problem below the existing left abutment spillway at Stewart Mountain Dam. Flow conditions in the impact area downstream of the auxiliary spillway were studied to prevent erosion damage that may endanger the spillway structure or the powerplant access road. Tests were made with gate openings of 3 ft, 15 ft, and fully open, representing discharges of 13,000, 53,000, and 94,000 ft^3/s, respectively. The test plan for determining the appropriate basin configuration included measuring velocities and pressures, and basic observation of the flow patterns. Each basin configuration was also photographed and video taped. (The video tape is available upon request).

Original plunge pool. - The original plunge pool consisted of a 4 to 1 slope downstream from the powerplant access road, El. 1444, to the basin floor at El. 1420. The floor began 116 ft downstream from the spillway and was 25-ft long with a 4:1 slope up to El. 1430 at the end of the basin. The right side of the basin was excavated to a 1-1/2 to 1 slope with the river channel forming the left side of the basin. The basin is shown in plan on figure 7. This basin was tested with the original 15° flip bucket.

Initial operation revealed the general energy dissipation characteristics of the flip bucket and basin. The small flip bucket deflection angle (9.3° above horizontal when combined with the chute slope) produced a flat jet impingement angle into the basin. No significant pressures were encountered in the basin for any discharge. Instead the jet swept through the basin, confined by the river channel on the left and the hillside on the right, producing undesirable flow conditions along the right bank. The small jet impingement angle into the basin did not dissipate much energy.

Flows for a 3-ft gate opening (Q = 13,000 ft^3/s) impinged on the upstream slope of the basin, then entered the tailwater where a hydraulic jump occurred. Downstream from the jump, velocities were highest along the right bank and decreased toward the river channel. A backflow occurred along the right bank adjacent to the hydraulic jump.

The discharge for a 15-ft gate opening (Q = 53,000 ft^3/s) almost entirely swept out of the basin with very little energy dissipation. The river channel controlled the jet on the left side of the basin and directed the flow back toward the right bank. This contained the high velocity flows near the right bank, causing the flow to climb the bank. Backflow still occurred adjacent to where the jet entered the basin.

At maximum discharge (Q = 94,000 ft^3/s), the jump was entirely swept out on the right side of the basin. Deeper tailwater in the river channel formed a weak jump and controlled the jet forcing it over toward the right bank. This concentrated the flow along the right bank, producing very high velocities along the entire right bank (figure 7). Backflows near the impingement area disappeared when the basin swept out. Maximum discharge in the basin is shown on figure 8.

Discharge sweeps through the basin, particularly on the right side, due to the small jet impingement angle and inadequate tailwater depth. This allowed very little energy dissipation. The concentrated flow along the right bank also produced unacceptably high velocities.

Recommended flip bucket and plunge pool design. - The recommended flip bucket design consists of a 35° (above horizontal) flip bucket formed by a 45-ft radius.

The 35° flip bucket angle provided two main advantages:

- A steeper impingement angle producing less tendency for the jet to sweepout of the basin and greater energy dissipation

- Moving the jet impact area further downstream, providing more protection for the end of the spillway

The 35° flip bucket in conjunction with the final plunge pool configuration greatly improved flow conditions.

In the several basin modifications which were tested prior to determining the final configuration, the hillside which protruded into the basin was used in an attempt to force a hydraulic jump in spite of inadequate tailwater depth. This approach was abandoned and these recommended plunge pool modifications adopted:

- Lower the basin floor to El. 1410.

- Steepen the upstream slope from the powerplant road (El. 1444) to the basin floor to 1-1/2 to 1.

- Excavate the entire right side of the basin on a 3/4 to 1 slope from El. 1450 to the basin floor at El. 1410. Continue this excavation about 250 ft downstream where the slope meets the original topography. The recommended plunge pool basin is shown on figures 5 and 6.

The steeper upstream slope prevented impingement on the slope for all discharges after the jet swept out of the chute. The hydraulic jump in the chute during initial gate opening will cause flow over the end of the bucket. This flow impinges on the powerplant road and down the 1-1/2 to 1 slope upstream of the basin. This flow condition will require protection for the roadway and possibly the slope, depending on the condition of the rock.

Removing the protruding hillside on the right side of the basin greatly improved flow conditions. Flow patterns were less turbulent with a uniform hydraulic jump formed for lower discharges and more evenly distributed flow for higher discharges. The river channel along the left side of the basin still partially restricted the spread of the jet on this side, but did not force the flow toward the right bank as had previously occurred. No backflows occurred along the right embankment for any discharge.

For a 13,000 ft^3/s discharge, the jet entered the basin at the base of the 1-1/2 to 1 slope forming a hydraulic jump (figure 9). Maximum velocities downstream of the jump were 12.3 ft/s 150 ft downstream, 8.4 ft/s at 250 ft, and 10.6 ft/s at 350 ft. The maximum pressure, equivalent to 30 ft of water, was measured at the intersection of the 1-1/2 to 1 slope and the basin floor.

For a 53,000 ft^3/s discharge, the jet entered the basin about 130 ft downstream from the flip bucket (figure 10). A hydraulic jump still formed but was close to sweepout. Maximum velocities downstream of the jump were 22.5 ft/s at 250 ft, and 24.5 ft/s at 350 ft. The maximum pressure of 35 ft was measured about 164 ft downstream from the flip bucket.

The jet at maximum discharge entered the basin about 130 ft downstream of the flip bucket (figure 11). The jet impinged on the basin floor and swept across the floor for about 50 ft before a hydraulic jump formed. Maximum velocities downstream of the basin were 34.4 ft/s at 150 ft, 29.9 ft/s at 275 ft, and 39.9 ft/s at 400 ft downstream. Maximum velocities were not reduced by this basin modification but were more evenly distributed than with previous configurations. Flow across the width of the basin was more uniform, with waves producing only slight overtopping of the right bank at El. 1450 about 300 ft downstream.

Maximum pressures, produced by maximum discharge, were measured on the basin floor about 165 ft downstream from the flip bucket. For tailwater El. 1443.8 (combined spillway maximum discharge) the maximum pressure was 46 ft, only 12.2 ft above the basin tailwater. For tailwater El. 1432.4 (no discharge from the existing spillway) the maximum pressure was 54.4 ft, 32.4 ft above the tailwater. The jet will still erode a plunge pool basin with the depth depending upon the amount and length of discharge and the integrity of the rock in the basin. With the uniform cut slope along the right bank the erosion process should be more predictable and less damaging. Erosion damage below the spillway should not produce any excessive problems based upon the geologic data from the site, which indicated that the rock in the basin area is not fractured and has good strength.

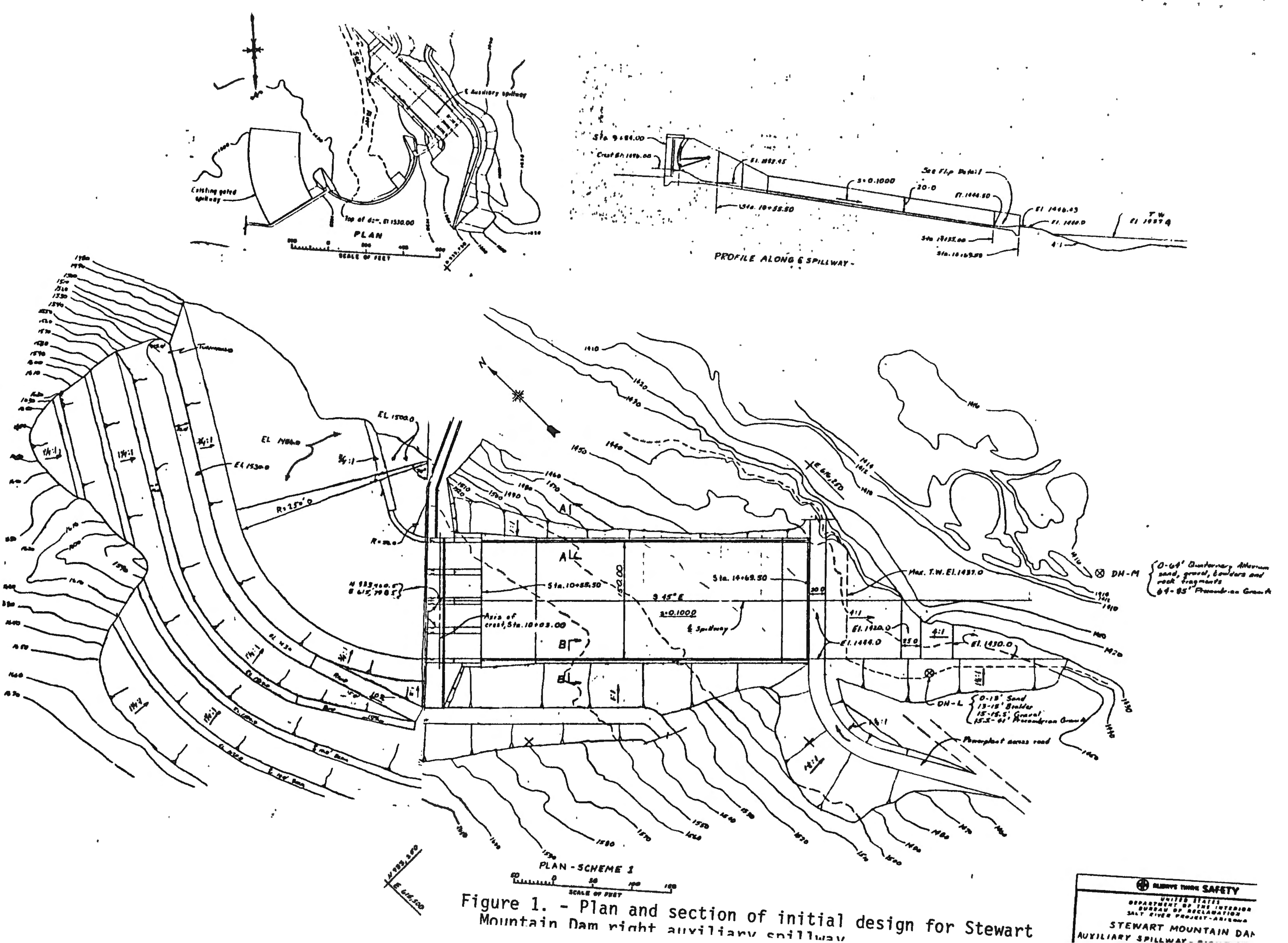

Figure 1. - Plan and section of initial design for Stewart Mountain Dam right auxiliary spillway.

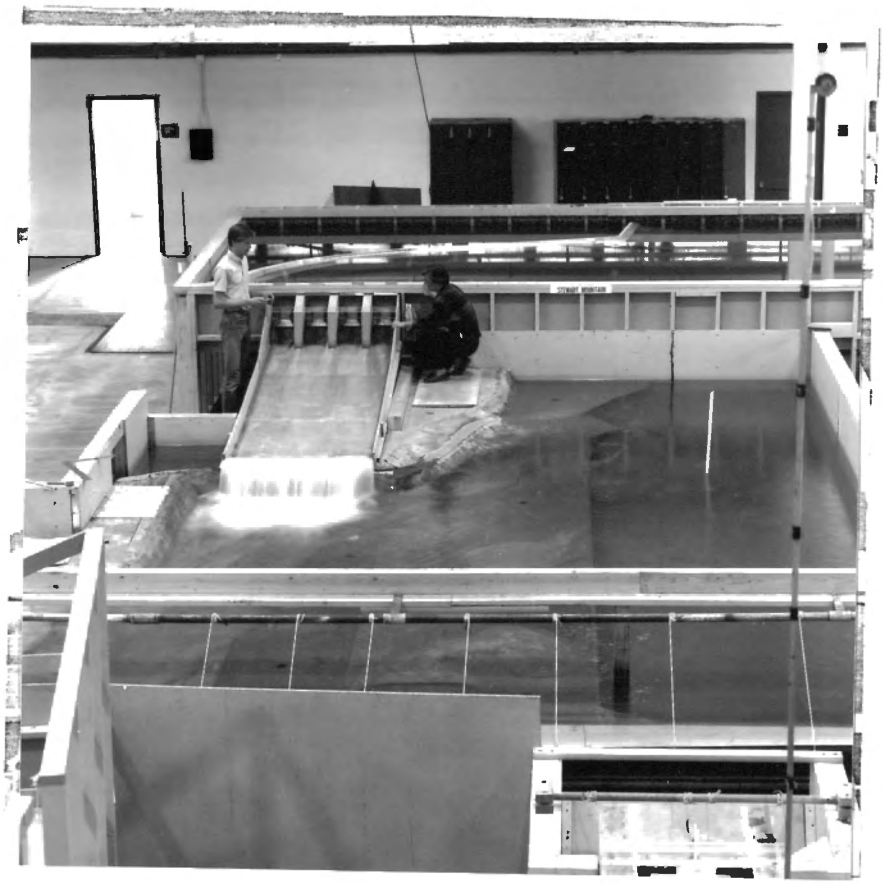

Figure 2. - Overall view of the 1:40 scale model - Stewart Mountain Dam auxiliary spillway.

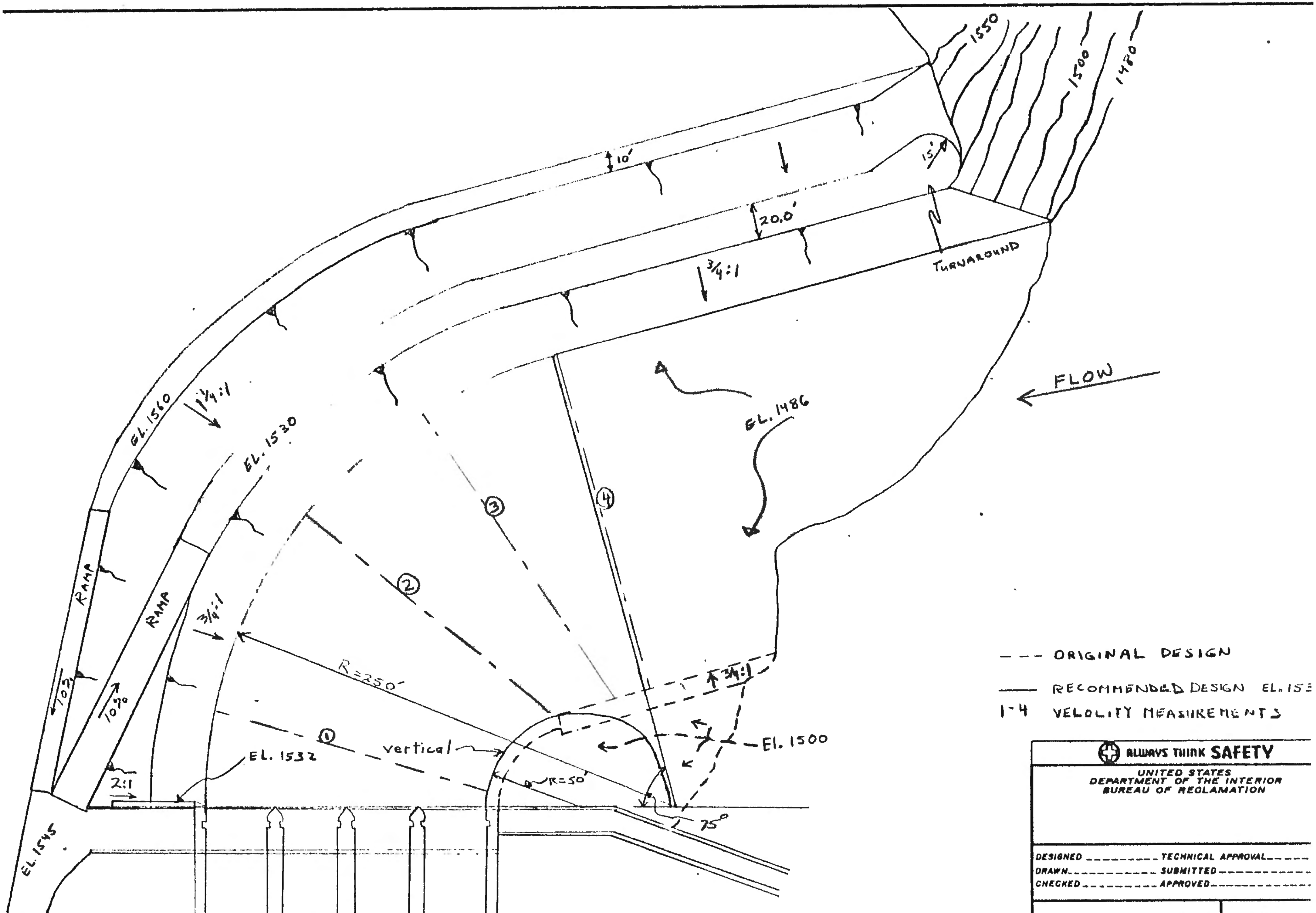

Figure 3. - Approach channel geometry.

(10½"x18")

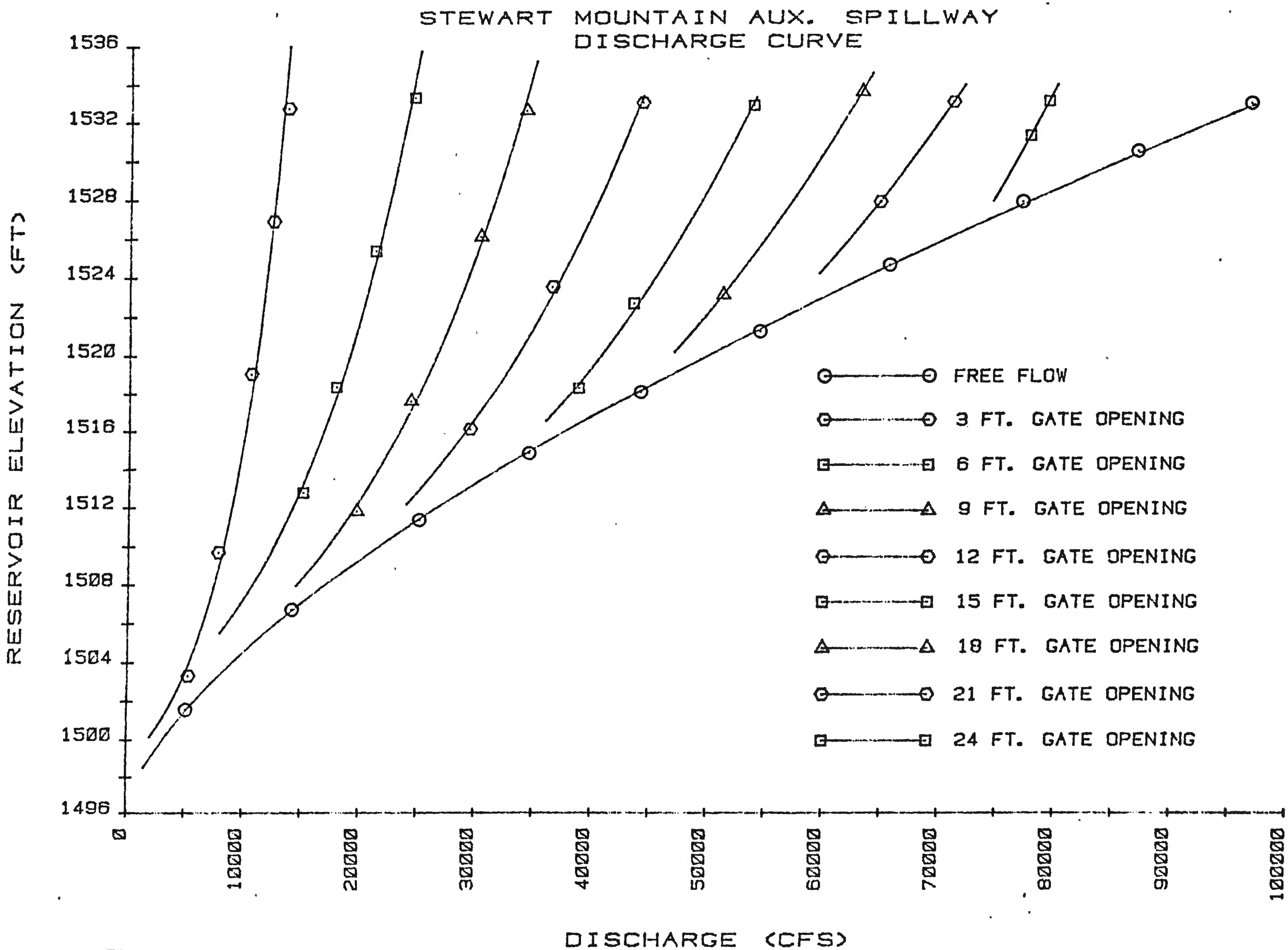

Figure 4. - Right spillway discharge curves for equal gate operation, 3-ft gate opening increments.

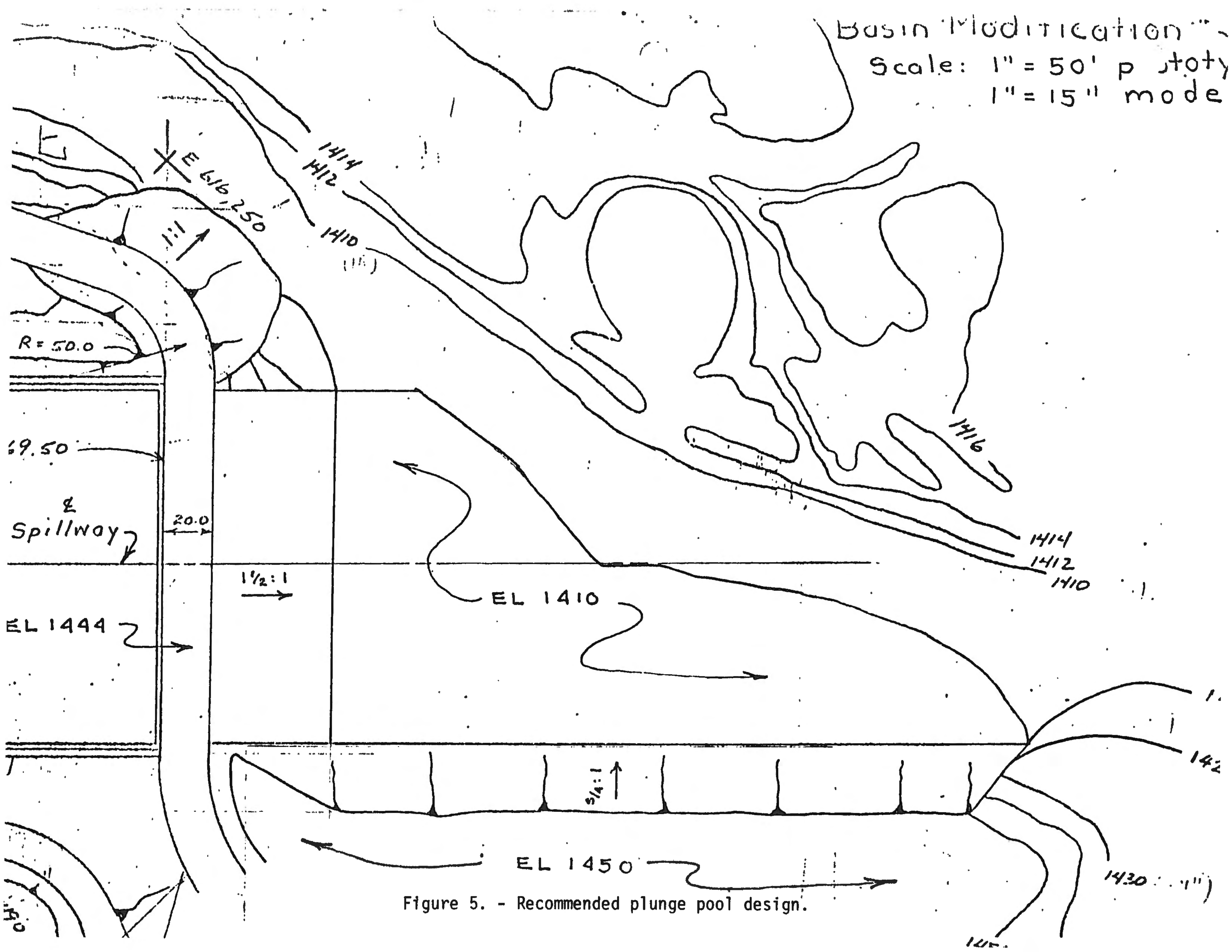

Figure 5. - Recommended plunge pool design.

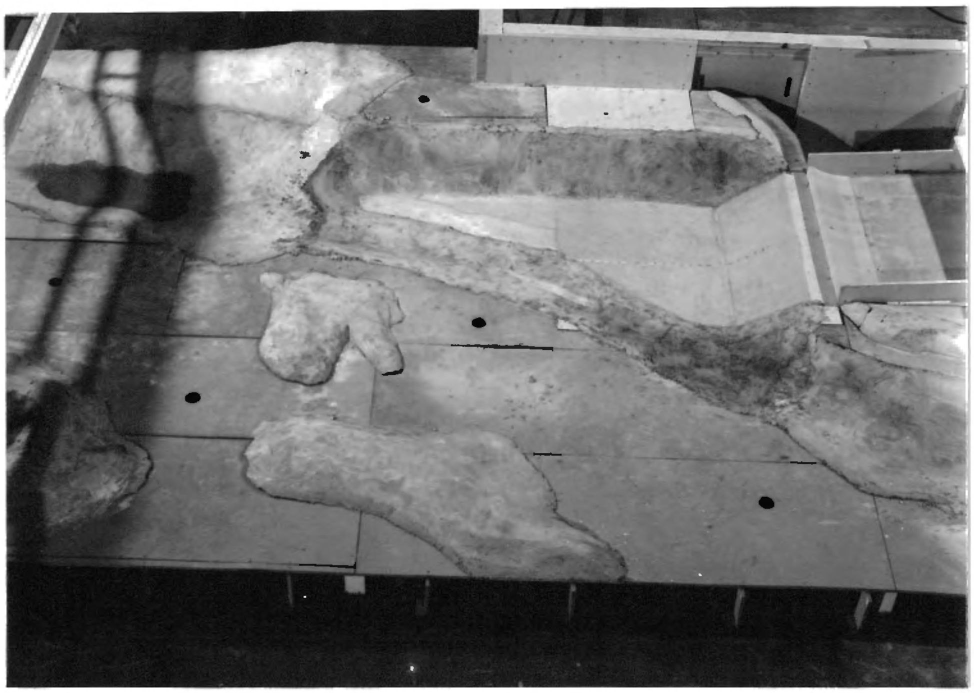

Figure 6. - Recommended plunge pool design.

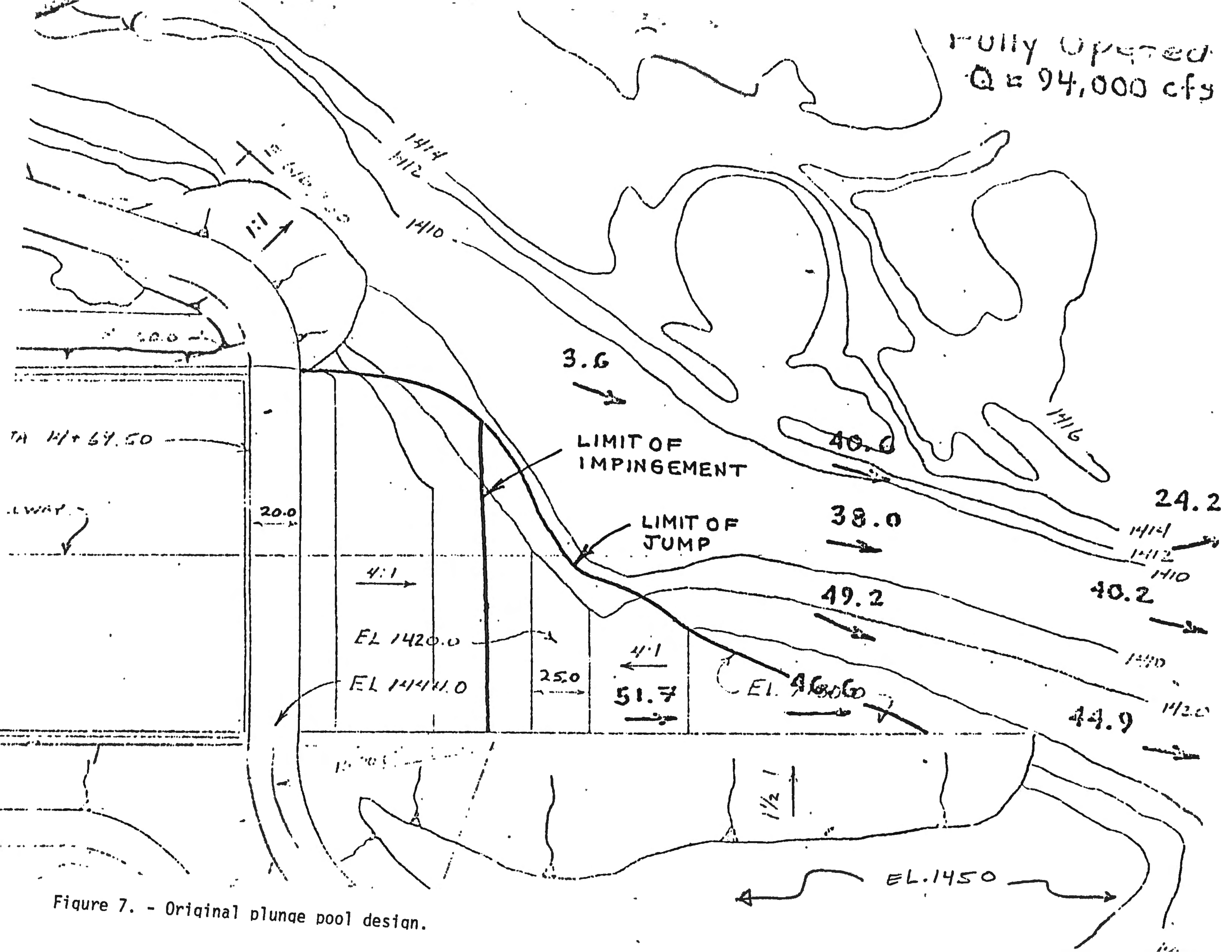

Figure 7. - Original plunge pool design.

Figure 8. - Original plunge pool design operating under maximum discharge.

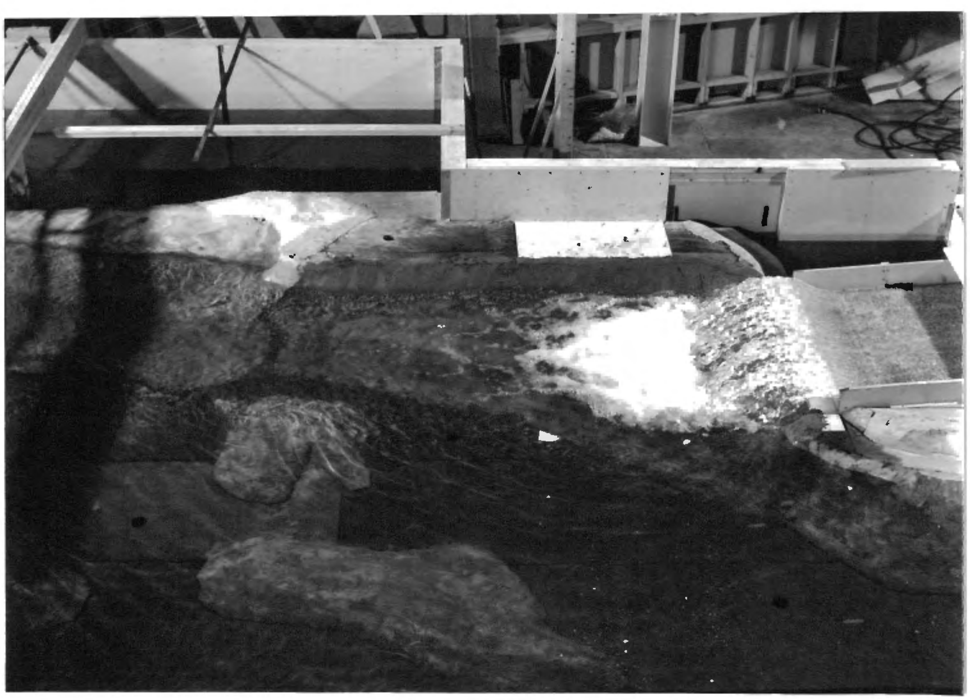

Figure 9. - Recommended plunge pool design - Q = 13,000 ft^3/s.

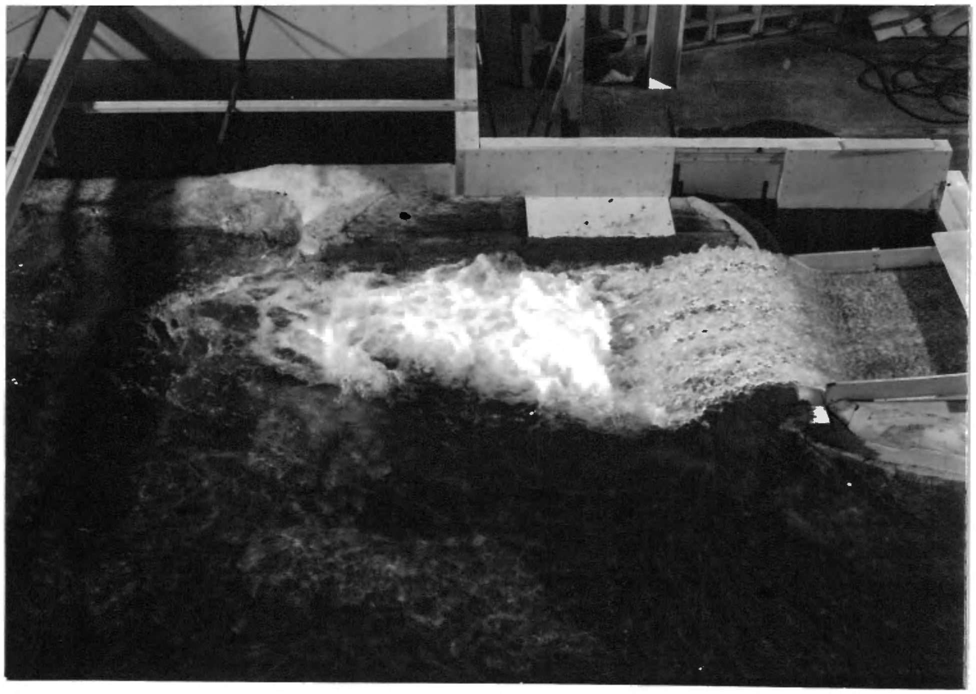

Figure 10. - Recommended plunge pool design - Q = 53,000 ft^3/s.

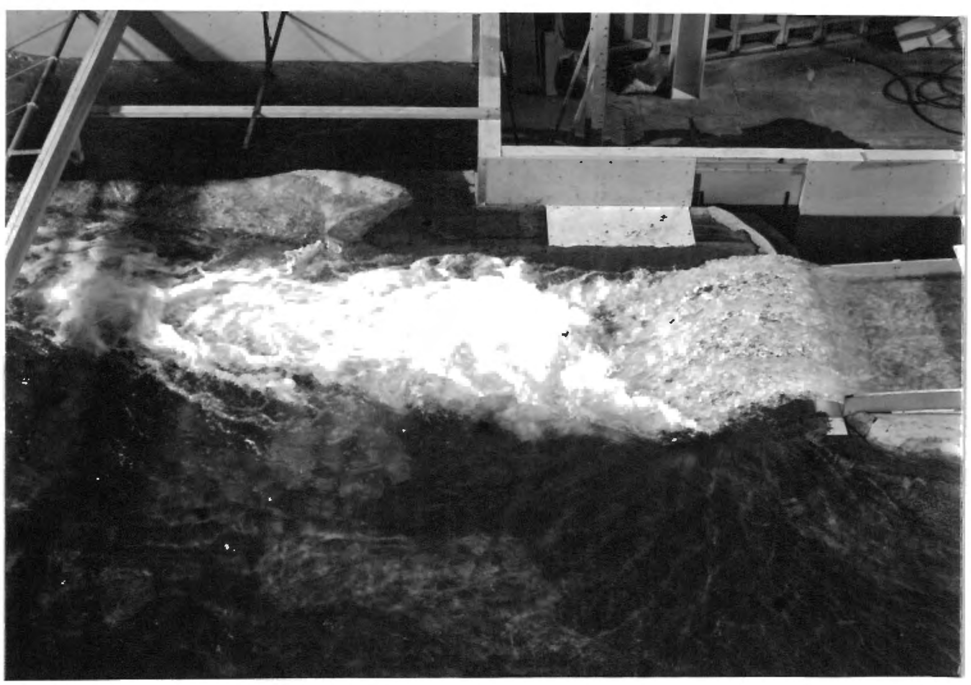

Figure 11. - Recommended plunge pool design - Q = 94,000 ft^3/s.