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HYDRAULIC MODEL STUDIES OF BARTLETT DAM SERVICE AND AUXILIARY SPILLWAYS

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16. ABSTRACT

The Bureau of Reclamation evaluated modifications to the service spillway and the addition of an auxiliary spillway at Bartlett Dam. Passage of the updated PMF (probable maximum flood) for Bartlett Dam required these modifications. The service spillway studies were used to determine the maximum spillway discharge, flow profiles through the spillway, the loading on the chute, flip bucket, and walls. The auxiliary spillway studies determined the flow capacity of the fuseplugs, the approach and exit channel velocities and flow depths, and location of potential waste areas in the surrounding site.

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by

Kathleen Houston Frizell

Hydraulics Branch
Research and Laboratory Services Division
Denver Office
Denver, Colorado

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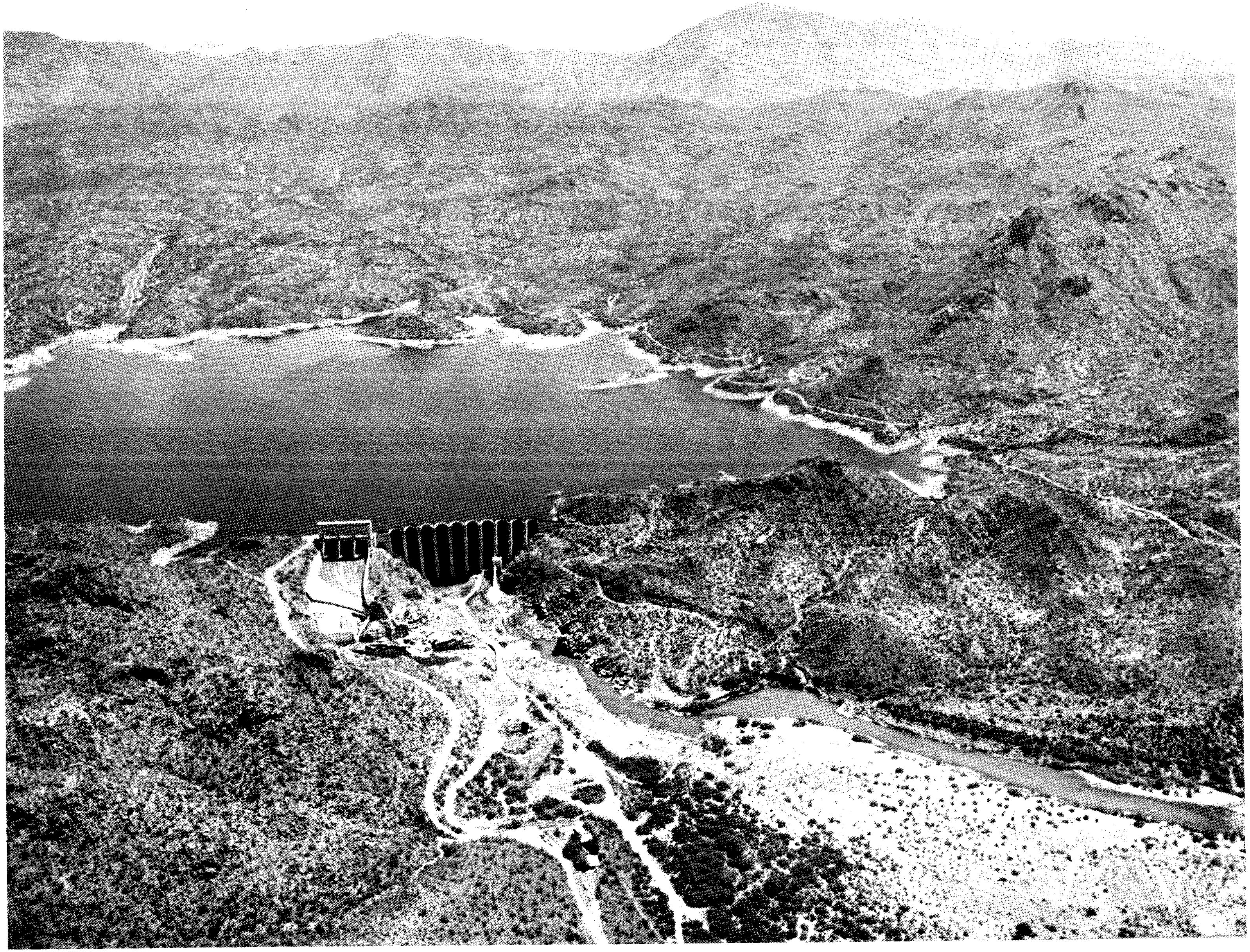

Bartlett Dam and Service Spillway (1981).

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PURPOSE

The hydraulic model studies were performed to determine the modifications necessary for the service spillway and to provide design information for the new auxiliary fuseplug spillway. The service spillway model was used to investigate flow ranges and conditions that will exceed the present design value of 175,000 ft^3/s. The auxiliary fuseplug spillway will be required to pass all flows exceeding the capacity of the service spillway and up to the PMF (probable maximum flood). The auxiliary spillway model provided design information by investigating the discharge and flow conditions through the fuseplug embankment sections.

INTRODUCTION

Bartlett Dam, located 48 miles northeast of Phoenix, Arizona, on the Verde River, was constructed by Reclamation between 1936 and 1939. Bartlett Dam is a multiarch concrete structure with a structural height of 287 feet and an 800-foot-long crest. The service spillway is located on the right abutment and is controlled by three 50- by 50-foot Stoney gates. The spillway chute is superelevated to the left with a flip bucket at the end. Spillway releases fall into a rock plunge pool with a massive cyclopean masonry block at the toe of the flip bucket (frontispiece). The design capacities of the service spillway and outlet works are 175,000 ft^3/s and 3,600 ft^3/s at reservoir elevation 1798, respectively. Bartlett Dam is operated by the SRP (Salt River Project).

The current PMF for Bartlett Dam has a peak of 562,000 ft^3/s, 2.37 times the previous PMF of 237,000 ft^3/s. To meet Safety of Dams requirements, Bartlett Dam will be modified to prevent overtopping and subsequent failure of the dam. The dam will be raised, the service spillway modified, and an auxiliary fuseplug spillway constructed in a reservoir saddle area south of the dam. Because Horseshoe Dam, located upstream on the Verde River, will also have a fuseplug spillway, the dams were treated as a river system when routing floods and determining impacts on downstream communities.

HYDRAULIC MODELS

Two models were constructed in Reclamation's hydraulic laboratory for complete investigation of the service and auxiliary spillways. The scope of work and the location of the hydraulic structures relative to one another required that separate models be built.

The Froude scaling law was used for scaling the models because gravitational forces predominate. The service spillway model was constructed to a 1:60 scale. The auxiliary spillway model was constructed to a 1:55 scale. Froude law similitude produces the following relationships for the models:

Length ratio	= L_r	=	60:1 (service spillway)
		=	55:1 (auxiliary spillway)
Velocity ratio	= $L_r 1/2$	=	$60^{1/2}$ = 7.75:1 (service spillway)
		=	$55^{1/2}$ = 7.42:1 (auxiliary spillway)
Discharge ratio	= $L_r 5/2$	=	$60^{5/2}$ = 27,885.48:1 (service spillway)
		=	$55^{5/2}$ = 22,434.00:1 (auxiliary spillway)

The hydraulic models were calibrated using the permanent laboratory Venturi systems to measure the inflows. A point gauge and/or conductance probe were used in the head boxes to record reservoir elevations.

The 1:60 scale service spillway model included the immediate reservoir approach channel, the dam access bridge across the spillway, the spillway crest with gate slots, and the superelevated spillway chute with the flip bucket. The extremely complex superelevated spillway chute had to be formed from templates developed from the as-built drawings. The entire spillway was constructed using high-density urethane foam that could be shaped to match the templates. No downstream topography was included. The issue of erosion potential in the service spillway plunge pool was analyzed empirically. An overall view of the service spillway model is shown on figure 1.

The 1:55 scale auxiliary fuseplug spillway model included about 1,015 feet of the reservoir with the curved approach channel cut upstream from the fuseplug control section. The fuseplug embankment sections were constructed using high density urethane foam. The fixed fuseplug sill or control section was modeled in plywood with the splitter walls modeled using urethane foam. The downstream topography was modeled from the fuseplug section to the river channel below for a distance of about 990 feet. The model topography was constructed using fixed bed material. An overall view of the model, with the originally designed auxiliary fuseplug spillway, is shown on figure 2.

SERVICE SPILLWAY RESULTS

The following results were obtained from the 1:60 scale service spillway model:

- The spillway rating for gated discharges up to the previous design value of 175,000 ft^3/s has been well documented (Burgi, 1981). With the spillway gates fully open and the reservoir surface rising, flow changes from free flow to full orifice control by the top-of-dam bridge upstream from the gate structure. The service spillway is capable of passing 287,535 ft^3/s at reservoir El. 1820.80 (fig. 3) with the dam height raised, the spillway gates fully open, and orifice flow produced by control with the top-of-dam bridge.

- Pressure profiles were measured on the upstream vertical face of the top-of-dam bridge to determine loading conditions for design (fig. 4). The profiles correspond to expected hydrostatic loading, except near the bottom edge where flow through the spillway opening slightly decreases the pressure.

- Pressures were measured along the invert of the spillway chute centerline beginning at Sta. 2+50 and continuing through the flip bucket. Pressures were also measured on the invert at the base of the left wall beginning at Sta. 4+91.97 and continuing through the flip bucket. Pressures were also measured up the left wall in the flip bucket at Sta. 5+93.5. Invert pressures were also measured at the base of the right wall through the flip bucket. Pressures along the invert of the chute centerline were not excessive and showed only one discontinuity because of the rapidly changing geometry near Sta. 4+00 (fig. 5). The flip bucket invert and wall pressures were lower than calculated theoretically because of the large flow depths when compared to the bucket radius (figs. 6 and 7).

- Water surface profiles measured vertical to the chute along the chute centerline and the walls are shown as cross sections vertical to the centerline station on figure 8. These profiles show the flow depths through the chute and the locations of wall overtopping for the existing wall heights. Figure 9 shows the water surface profiles along the left and right walls, with the locations of wall overtopping shown on an arbitrarily chosen wall height.

- An analytical assessment of further plunge pool erosion was made. The jet trajectory calculations were based upon model data of the flow depths at the end of the spillway flip bucket (fig. 8g). Calculations determined that further erosion would occur under the PMF; however, the trajectory of the jet would produce damage farther downstream than previously experienced. The assessment, therefore, determined that further erosion would not endanger the dam and would not be addressed at this time under Safety of Dams funding.

AUXILIARY SPILLWAY RESULTS

The model was initially designed and constructed with 4 fuseplugs (Pugh, 1985) and tested with a maximum water surface of El. 1825 based upon the VE (Value Engineering) team recommendation. Amid the testing program, an investigation of the dam stability by the Concrete Dams Branch determined that the raised dam would be stable only up to reservoir water surface El. 1820.80. To prevent the reservoir water surface from exceeding El. 1820.80, changes were made to the fuseplug sections using initial data from the model. These preliminary results indicated that the discharge coefficient was higher than the broad-crested weir coefficient used during feasibility design. The number of fuseplugs was decreased from four to three. The total length of the control section was narrowed and length of the first two fuseplug sections was increased to pass more flow early in the flood hydrograph. The results for the initial designs are given in full. The final design results are given where they vary significantly from the previous results.

The following results were obtained from the initial 1:55 scale auxiliary spillway model geometry:

- The discharge capacity was determined for each fuseplug section failing sequentially, until the section was flowing entirely open. The family of discharge curves for 1, 2, 3, and 4 fuseplugs failed is shown on figure 10. At reservoir El. 1825, the fully open fuseplug sections passed 356,000 ft^3/s. This discharge was greater than that needed to pass the PMF. The discharge coefficient for the fully open section with a net bottom width of 350 feet varied from 2.63 to 3.10.

- Flow through adjacent fuseplugs produced significant drawdown at the axis of the fuseplug section, resulting from the long, flat approach channel. Drawdown within the approach channel produces a water surface at the pilot channel of the next fuseplug to fail that is lower than the reservoir water surface. Therefore, the reservoir water surface will be higher than expected before the next fuseplug fails and must be accounted for in the flood routings.

- Velocities were measured through the approach channel to assist with determining erosion potential of the channel invert and cut slopes. The measurement locations are shown on figure 11a. Average velocities at each measurement cross section and through the failed sections are shown for 1, 2, 3, and the fully open section in the form of bar graphs (fig. 11). Velocities were highest along the right wall of the curved approach channel looking upstream from the open sections. Minimal erosion is expected based upon the model results, a tractive force analysis, and the fact that the excavation is in rock.

- Velocities were measured adjacent to the splitter walls on the face of the next fuseplug to fail. The velocity contours (fig. 12) were used to determine if the splitter walls extended far enough upstream from the embankment to allow the use of about 2-foot riprap protection on the upstream face of the fuseplugs. The splitter wall adjacent to the fourth fuseplug was extended to 5 feet normal to the embankment slope (fig. 12d), which reduced all velocities to an acceptable level.

- Loading on the splitter walls will be determined from the water surface profiles measured through the fuseplug control section. The profiles were developed for the maximum water surfaces just prior to failure of the adjacent fuseplug. Until all the fuseplugs had failed, these profiles showed a severe contraction around each splitter wall on the right side of the failed section. Upon failure of all the fuseplugs, the curved approach channel produced only minimal contractions. A typical flow condition is shown on figure 13 with two failed sections.

- Velocities measured adjacent to the footing of the high voltage transmission tower located in the reservoir upstream from the approach channel indicated that significant erosion near the footing should not occur.

- Velocities, approaching 53.5 ft/s, measured downstream from the fuseplug section were used to determine the possibility of headcutting from the downstream area back up to the cutoff wall at the downstream edge of the fuseplug section. The velocities and the contours of the topography indicated that headcutting would occur, but not to the extent that would undermine the cutoff wall allowing drainage of the reservoir to a level lower than 1780.

- A dike about 750 feet long should be constructed downstream from the fuseplug section from the end of the 3/4:1 cut slope towards the high knoll to the right of the section. This dike will direct flows from the fuseplugs through the main drainage channel to the river channel, preventing water from flowing down a natural drainage area to the right of the fuseplug section (fig. 14).

- Excavated material placed to the left of the approach channel immediately upstream from the control section above the cut slope is actually beneficial for higher releases. Material may also be disposed of in the reservoir finger that approaches the control section, but this material must be limited in the upstream direction to the end of the 800-foot radius of the approach channel cut. Any material placed in the reservoir or remaining after excavation must be below the level of the control section at El. 1780.

The following results were obtained from the final auxiliary spillway geometry tested:

- The family of discharge curves for the sequential opening of the fuseplug section is shown on figure 15. At reservoir El. 1820.80, the fuseplug section passes 265,250 ft^3/s. The discharge coefficient for the fully open section with a net bottom width of 330 feet varied from 2.50 to 3.08 (fig. 16). (The final fuseplug widths are slightly greater than those tested, and should pass the additional 9,200 ft^3/s needed to pass the entire PMF.)

- Velocity measurements in the approach channel and at the fuseplug axis were retaken for the three fuseplug sections. Figure 17 shows that the same flow trends exit, but with slightly varied results caused by the change from four to three sections. The velocities do not indicate excessive erosion would occur in the unprotected approach channel such that the concrete sill would be undermined.

- Velocities remeasured on the face of the next fuseplug to fail (fig. 18) indicated velocities were at an acceptable level for protecting with riprap.

- The depths through the control section were measured to determine the loading on the splitter walls and to verify the location of critical depth (fig. 19).

OPERATING CRITERIA

Bartlett Dam is located just downstream from Horseshoe Dam on the Verde River northeast of Phoenix, Arizona. Both dams are being modified, primarily under Safety of Dams funding, by adding auxiliary fuseplug spillways to pass PMF flows. The dams are being treated as a river system for controlling flows because both dams will have fuseplug spillways and the flood routing through Horseshoe Dam affects Bartlett Dam. The criteria for sizing the fuseplugs was that the river stage at Fort McDowell, the first community downstream from Bartlett Dam, would not be more than 1 foot higher than the stage predicted if the dams had never been constructed.

With arrival of flood flows to Bartlett Dam, the service spillway will be operated as needed, and will pass about 213,000 ft^3/s at reservoir El. 1803, before the reservoir rises enough to fail the first fuseplug. None of the fuseplug failures will cause the downstream criteria to be exceeded. Both spillways operating to their full capacities pass the PMF of 562,000 ft^3/s at reservoir El. 1820.8.

SERVICE SPILLWAY INVESTIGATIONS

Test Plan

The test plan consisted of gathering data needed to confirm the ability of the service spillway to pass the flow and withstand the loading associated with the new PMF. Therefore, the service spillway was tested only from the existing maximum reservoir water surface elevation of 1798 and discharge of 175,000 ft^3/s up to a surface elevation of 1825. The model study was conducted because the complicated spillway chute geometry (fig. 20) did not allow existing water surface profile programs and routine loading analyses to be conducted analytically.

All pressure data were recorded at four reservoir elevations: 1798, 1815, 1820, and 1825. Final interpolations were made for the eventual maximum water surface of 1820.80. Water surface profiles were recorded for reservoir water surface El. 1825 and checked for the final water surface at El. 1820.80.

The following design information was obtained from the service spillway model:

- Discharge rating curve and flow conditions as the flow transitions from free flow to orifice flow underneath the top-of-dam bridge for use in flood routings.

- Pressure measurements on the vertical upstream face of the top-of-dam bridge for determining the structural design of the bridge and parapet.

- Pressure measurements on the invert of the chute through the flip bucket to determine structural integrity of the chute. These measurements were taken along the chute centerline through the bucket, on both the right and left sides of the bucket near the base of the walls, and in the low point of the superelevated chute just upstream from the P.C. (point of curvature) on the left side of the bucket.

- Pressure measurements were also recorded on the left wall at third points up the wall to determine the wall loadings through the flip bucket.

- Water surface profiles were measured normal to the chute centerline at several locations down the chute and also parallel to the left and right chute walls. This information was used in determining necessary wall heights and computing the jet trajectory.

Testing began in March 1992 and was completed in June 1992.

Discharge Rating and Flow Conditions

The spillway rating for gated discharges is given in a previous model study report (Burgi, 1981). The model investigated the discharge capacity and flow conditions that exceeded gated flows and extended to flows of the maximum proposed reservoir water surface El. 1825. The dam and bridge parapet will be modified to prevent overtopping of either the dam or the roadway upstream from the spillway gate structure. The parapet on the upstream side of the bridge will be raised to El. 1824.5. The vertical parapet and bottom of the bridge, El. 1801, form a short box culvert upstream from the spillway gate structure.

The service spillway is capable of passing 287,535 ft^3/s at reservoir El. 1820.80 (fig. 3). The flow changes from free flow, with the gates fully open, to full orifice control by the top-of-dam bridge upstream from the gate structure, as the reservoir water surface rises. During free flow, the following equation may be used to compute the discharge for a given head above the crest:

$$Q = 523.1215 H^{3/2}$$

This equation should be used for heads over the crest from 50 feet (El. 1798) to 55 feet (El. 1803). (This equation gives 185,000 ft^3/s for the discharge at reservoir El. 1798. The original design required 175,000 ft^3/s be passed at El. 1798. Previous studies (Burgi, 1981) also gave this larger flow at El. 1798.)

The square upstream edge of the bridge begins controlling the flow through the center spillway bay at reservoir El. 1803 and a discharge of 213,376 ft^3/s. Drawdown occurs around the two 30-foot-radii side piers forming the spillway approach. At reservoir El. 1807 and a discharge of 232,362 ft^3/s, the center bay is operating under full orifice control with the left and right bays flowing freely. Between reservoir Els. 1807 and 1811, the left and right bays begin changing to orifice flow with only the outside portions of each bay still drawing air under the bridge. While the flow is transitioning from free flow to orifice flow from 55 feet (El. 1803) to 63 feet (1811), the following equation may be used to compute the discharge:

$$Q = 5167.96H - 72548.17$$

The bridge fully controls the flow at reservoir El. 1811, producing a discharge of 251,940 ft^3/s. As the reservoir rises to El. 1820.80, under full orifice control, the discharge increases only slightly to 287,535 ft^3/s. During full orifice control, from 63 feet (El. 1811) to the maximum water surface (El. 1820.8), the following equation may be used to compute the discharge:

$$Q = 59819.81\sqrt{H} - 222865$$

Flow Conditions and Pressures on the Spillway Bridge

A vertical wall will be added on the upstream side of the service spillway bridge to prevent dam overtopping under the maximum water surface El.1820.80. The bridge has a square-edged bottom on the upstream side at El. 1801 across the spillway. The upstream face of the bridge wall is located 13.79 feet upstream from the crest apex (fig. 20). The bridge spans the spillway upstream from the gate structure and will control flows through the spillway as the reservoir water surface rises. The effect of the bridge on control of the water surface and the loading on the bridge were determined with the model.

Fillets, installed in the gate slots of the model and spanning each bay, represented the bottom of the gates in their fully raised position. Flow separated off of the square upstream edge of the bridge, producing a water surface below the level of the fully raised gates. This flow condition prevented gate vibration and possible damage caused by additional loading.

Piezometer taps were located on the centerline of the spillway up the vertical face of the bridge wall to determine loading conditions for the structural design of the bridge. Figure 4 shows the loading in feet of water on the vertical face of the bridge for water surface elevations of 1815, 1820, and 1825. The loading conditions verify typical water loading profiles, except near the very bottom of the bridge where the flow underneath the bridge reduces the pressure just above the corner. The center spillway bay of the bridge parapet experienced loading sooner than the outside bays as described in the previous section. The amount of loading, however, will be similar across the bridge parapet, under the design flow rate and water surface elevation.

Spillway Chute Invert and Wall Pressures

Invert pressures were measured on the centerline of the spillway chute on 30-foot increments. The measurements began at the spiral curve at Sta. 2+50 and continued to just upstream from the P.C. of the flip bucket. Flip bucket invert pressures were measured at four locations. Invert pressures were also measured on the left side of the spillway chute from Sta. 4+91.97 through the flip bucket and on the right side through the flip bucket. Wall pressures were measured at the two-thirds point in the flip bucket on the left wall. The locations for the piezometer taps are shown on figure 21. Data were recorded for reservoir Els. 1798, 1815, 1820, and 1825.

The centerline chute invert pressures are shown on figure 5. The profiles indicate the loading on the surface of the spillway chute. A slight discontinuity in the pressure profiles occurs at about Sta. 4+00. This discontinuity is probably caused by the change in the chute geometry from a spiral through Sta. 4+00, to a 600-foot radius for 25 feet, to a 500-foot radius from Sta. 4+25 through the end of the chute. As expected, the centrifugal force through the flip bucket causes a pressure increase, which returns to hydrostatic pressure near the end of the bucket (figs. 5 and 6). These bucket pressures are, however, much less than those predicted by theoretical computations because of the large flow depths compared to the bucket radius. Design guidelines (Peterka, 1978) state that the bucket radius should be about four times the flow depth, but in this case the bucket radius is about 1.25 times the flow depth. Therefore, the bucket is not long enough to turn the jet, and the full computed centrifugal force will not be seen by the bucket invert. Pressures were measured on the left side of the chute invert in the low point of the superelevated chute. Pressures (fig. 6b) along the base of the left wall do show increased flow depths in this area.

Pressures were also measured at three locations up the left wall of the flip bucket at Sta. 5+93.5 or about 0.7 times the bucket length from the beginning of the bucket (fig. 7). Literature has shown that wall pressures should be at a maximum at this location. Again, theoretical calculations indicated greater pressures than were actually measured because of the small bucket radius compared to the flow depth.

Chute Water Surface Profiles

Earlier model studies had indicated that the right spillway chute wall would overtop at a flow rate of about 240,500 ft^3/s at reservoir El. 1805.8. Because the flow rate and reservoir elevation will exceed this amount during the PMF, concern arose regarding the locations and amounts of chute wall overtopping. Overtopping of both left and right walls, during the PMF, is shown on figure 22. The walls were then raised in the model to contain the entire flow and water surface profiles recorded for reservoir El. 1822 and a flow rate of 291,724 ft^3/s. This elevation was the predicted maximum allowable reservoir water surface, at the time of the study, based upon stresses in the dam.

Water surface profiles were measured vertical to the chute centerline at the same stations as the pressures were recorded. The water surfaces across the chute were determined using an electronic probability probe, attached to a point gauge with a vernier, that measures a fluctuating water surface. The probe reads zero when not in contact with the water and 100 when fully submerged. The average water surface, therefore, occurs at a reading of 50, or when the probe is submerged 50 percent of the time. The water surface was measured at the centerline and vertically up from the intersection of the walls and the chute invert. The fins in the chute formed by the two spillway bay piers were not accounted for in the measurements. Water surfaces along both walls were simply marked along the raised walls.

Flow began splashing over the right wall at a discharge of 243,810 ft^3/s and flow began overtopping the left wall at 266,780 ft^3/s. The discharge where right wall overtopping begins compares well with the data from the 1981 model study. Right wall overtopping begins sooner than left wall overtopping because of the chute curvature into the flow on the right and away from the flow on the left. Overtopping of the right wall began at Sta. 2+65 and continued through Sta. 4+20 with the maximum overtopping of 4.3 feet occurring at Sta. 3+25 (fig. 9b). Overtopping of the left wall began at Sta. 3+10 and continued through Sta. 4+15 with the maximum overtopping of 3.6 feet at Sta. 3+40 (fig. 9a). Table 1 shows the flow depths measured at each station and the corresponding wall overtopping depths. Sections of each water surface profile are shown on figure 8 with the water surface shown in relation to the height of the existing spillway chute walls.

The final chute water surface elevations were determined to be essentially the same under the eventual maximum reservoir El. 1820.80. Profiles remeasured on the walls showed the same elevations but shifted slightly downstream from the previous locations. The designers will use this information to design the appropriate chute wall heights to prevent overtopping of the walls during the PMF.

Jet Trajectory Measurements and Calculations

Data were gathered from the model to determine the jet trajectory under the PMF. The trajectory identified the impingement area, thus the expected location for initiation of erosion in the plunge pool under the PMF. For the trajectory calculation, nine depth measurements were recorded across the end of the flip bucket section (centerline Sta. 5+35). The velocity was determined from the cross-sectional area and the flow rate and was used in the equation for a projectile. The streamlines for the trajectory ranged from 224 feet on the right side of the chute to 194 feet from the left side. The result of this calculation showed that the jet would impact about 30 feet farther downstream than that of the previous design flow rate of 175,000 ft^3/s.

Table 1. — Service spillway chute wall overtopping locations and depths.

Station	Left side wall			Right side wall		
	El. (ft)	Depth (ft)	Overtopping (ft)	El. (ft)	Depth (ft)	Overtopping (ft)
1+88.12	1765.53	27.67	0	1766.93	29.07	0
2+50	1765.56	28.23	0	1767.52	29.07	0
2+65	1766.56	31.31	0	1769.31	32.42	0
2+80	1765.25	32.42	0	1769.87	33.54	1.8
2+95	1761.23	31.31	0	1768.58	32.98	2.5
3+10	1758.53	31.86	0	1768.35	33.54	4.3
3+25	1754.58	31.58	1.7	1766.38	32.42	4.3
3+40	1750.41	31.58	3.6	1764.30	31.31	4.1
3+55	1743.88	29.63	3.5	1762.26	30.19	3.2
3+70	1737.19	27.95	3.4	1760.17	29.07	2.0
3+85	1729.34	25.72	2.2	1758.27	28.23	0.9
4+00	1723.14	25.16	2.1	1757.74	28.51	0.8
4+15	1715.97	22.92	0	1758.84	30.19	0.8
4+30	1711.98	22.36	0	1758.02	29.91	
4+45	1709.26	22.08	0	1760.08	32.42	
4+60	1706.92	21.52	0	1761.87	34.66	
4+75	1705.93	21.80	0	1763.21	37.45	
4+90	1703.94	20.68	0	1764.12	39.20	
5+05	1703.12	20.68	0	End of right side of flip bucket is Sta. 4+90		
5+20	1701.74	20.13	0			
5+35	1700.92	20.10	0			
5+50	1702.91	22.92	0			
5+65	1704.89	25.72	0			
5+80	1709.12	30.75	0			
5+95	1711.10	33.54	0			

End of left side of flip bucket is Sta. 6+00

Plunge Pool Erosion Potential

Historical spillway releases at Bartlett Dam have produced significant erosion migrating back towards the toe of the chute and numerous attempts at erosion protection. The existing protection is a massive cyclopean masonry gravity section that has been in place since the early 1980's and was designed to protect against the potential erosion from flows up to 175,000 ft^3/s. The question was raised about the potential for further plunge pool erosion below the service spillway under the PMF which would require passage of 287,535 ft^3/s through the service spillway. Plunge pool flow conditions were not, however, to be investigated by the hydraulic model.

The design team determined that the criteria for acceptable erosion was any amount of erosion that would not cause failure of the dam. The design team decided that the dam could only be endangered if erosion could migrate upstream through the spillway chute and fail the right abutment of the dam. The dam failure could only be initiated if the plunge pool erosion undermined and failed the cyclopean masonry block at the toe of the chute. The following methods for determining the potential for plunge pool erosion under the PMF were investigated:

- Empirical determinations using available published literature on prediction of plunge pool erosion.
- A new empirical method of determining erosion potential currently being developed using stream power concepts applied to classified rock types.
- Analysis of historical releases and erosion damage with trajectory calculations from the existing spillway chute. This analysis includes topographic mapping of historical damage and rock types with judgment on location for scour potential under the new PMF.

All of the empirical methods require judgment about the location and extent of any predicted erosion. Thus, even the best available empirical methods still require sound engineering judgment in the final analysis.

Using these methods, the design team determined that erosion would occur. Plots of the historical erosion damage show that the cyclopean wall and the present exposed rock has stopped migration of the erosion upstream toward the spillway. Therefore, for flows similar to those already experienced, up to about 100,000 ft^3/s, the erosion seems to have stabilized. For greater discharges, the trajectory will cause the jet to impinge farther downstream, thus reducing the likelihood of upstream migration of the erosion. The design team felt that erosion will probably deepen and lengthen the plunge pool in the downstream direction. Under the PMF, the jet will likely impinge near or on the downstream rock wall of the present plunge pool. This impingement will cause recirculation of flow in the plunge pool until the downstream wall of the plunge pool erodes. The recirculating flows would probably cause the majority of the erosion in the rest of the pool.

In summary, erosion of the existing plunge pool is likely to continue under the PMF, but not in a location that would endanger the spillway chute and eventually the dam. Based on this decision, the design team concluded that expanding the model investigations to include an erodible plunge pool would not warrant the additional cost. For further information on the empirical methods and historical mapping, see Technical Memorandum No. 3110-Bart10-92.

AUXILIARY SPILLWAY INVESTIGATIONS

Test Plan

Auxiliary spillway testing began with the initial feasibility layout of the fuseplug sections as shown on figure 23. The purpose of the four fuseplug sections was to pass the portion of the PMF flow that exceeds the capacity of the service spillway without creating excessive discharges downstream. The maximum reservoir water surface, as proposed by the VE (Value Engineering) Team, was El. 1825. All testing was conducted under the maximum possible water surface elevation for each failed fuseplug section prior to failure of the next section. For example, if two fuseplug sections had failed, then tests were conducted for flows through the open sections with the water surface set to the pilot channel of the third section at El. 1807. This setting produced the maximum possible velocities and depths for conservative designs.

The following design information was obtained from the model:

- Discharge rating and coefficients for failure of the fuseplug sections from left to right looking downstream.
- Water surface profiles, depths, and velocities through the fuseplug control section.
- Effect of drawdown caused by failure of fuseplug sections on the reservoir elevation at which subsequent fuseplug sections fail.
- Effects of flow through fuseplug sections on the adjacent sections.
- Velocities to assess erosion potential in the upstream and downstream fuseplug channel.
- Flow velocities near the support for the 345-kilovolt transmission line tower in the reservoir.
- Potential sites for disposal of excavated material.
- Necessity to direct flow from fuseplug releases to the downstream river channel through a desirable area.

Tests were completed for the initial fuseplug layout when dam stability analyses determined that the maximum reservoir water surface must be limited to El. 1820.80. With information from the previous model testing and subsequent flood routings, the number of fuseplug sections was reduced to three and the total section was slightly reduced in width. This configuration (fig. 24) was then tested in the model to determine the flow conditions with fewer fuseplug sections.

Testing began in January 1992 with the initial fuseplug configuration. Initial testing was entirely completed when the fuseplug configuration was changed. Testing of the final designs was completed in August 1992. In October 1992, the side slopes in the fuseplug approach channel and control section were determined to be more stable than originally planned. As a result, both the side slopes were changed to 1/2:1. Because these slopes reduced the flow area slightly, the bottom width of the three sections were increased. This geometry was not modeled because the results should not be significantly altered.

Initial Auxiliary Fuseplug Configuration

The model was initially constructed with the auxiliary fuseplug section as shown on figure 23. The approach channel was excavated to El. 1780 on an 800-foot radius with 3/4:1 side slopes. The total fuseplug section was 372.5 feet wide with four fuseplug sections separated by three splitter walls. The two left (looking downstream) fuseplugs were 80 feet wide and the two right fuseplugs were 95 feet wide. The 1-foot-deep pilot channels began at El. 1803 with embankment heights beginning at El. 1804, and each increasing by 2-foot increments to maximum elevations 1809 and 1810, respectively. The separating splitter walls had 1:10 side slopes and were 30 feet high, which corresponded to the height of the maximum fuseplug section. The downstream channel invert slopes away on a 5-percent grade and daylights into the existing topography. Data from this configuration were used to determine the final design. Testing was completed under maximum reservoir water surface El. 1825. The results for this initial fuseplug design are stated in this section. Variations required for the final design will be discussed in the next section.

Discharge rating. — Discharge rating curves were developed for the sequential failure of the four fuseplugs from left to right looking downstream. Discharges were measured in each failed or open section until the water surface rose to the elevation of the adjacent pilot channel of the next fuseplug (fig.10). The discharge at reservoir El. 1825 with all fuseplugs failed was 356,000 ft^3/s. The discharge coefficient at reservoir El. 1825 was computed to be about 3.10, based upon the net bottom width of 350 feet between the splitter walls. The discharge coefficient applies to the fully open portion of the auxiliary spillway channel.

Losses through the long, flat approach channel produced a significantly lower water surface in the approach channel compared to the reservoir water surface. Therefore, the reservoir water surface will be higher than expected before successive pilot channels will overtop and fail. With the first fuseplug failed, the second fuseplug will begin breaching at reservoir El. 1806.13 (+1.13 ft). With the first two fuseplugs failed, the third fuseplug will begin breaching at El. 1808.6 (+1.6 ft). With three fuseplugs failed, the fourth will begin breaching at El. 1811.76 (+2.76 ft). The depth at the axis of the fuseplug section is 16.86 feet less than the reservoir water surface when all fuseplugs have failed. The flood routing must be adjusted for these expected delayed fuseplug breachings.

The batter on the splitter walls was flattened slightly to ensure appropriate compaction of the fuseplug material next to the walls. The wall batter was changed to 0.2:1, which reduced the available flow area, thus the discharge. The discharge at reservoir El. 1825 with all plugs failed was 316,500 ft^3/s, about 39,500 ft^3/s less than the original design. This initial discharge rating information was used to determine the final design of the fuseplug sections.

Approach channel velocities. — Velocities were measured across the curved fuseplug approach channel at the toe of the fuseplugs, at two locations in the upstream channel along the channel radius, and at the farthest upstream point in the channel excavation (fig. 11a). Velocities were measured with an OTT propeller meter at 0.2 and 0.8 the flow depth or 0.6 the flow depth for shallow depths, and averaged for each location. Velocities were measured for each failed section with the reservoir at the pilot channel of the successive fuseplug and at El. 1825 when all fuseplugs had failed.

Velocities, in general, increased with discharge as the fuseplug sections failed from left to right. As the sections failed, high velocities at the upstream edge of the control sill indicated flow through the failed section with velocities quickly decreasing across the face of the remaining plugs. The velocities were high at the upstream toe of the next splitter wall or at location No. 3 with one plug failed, location No. 5 with two plugs failed, and location No. 7 with three plugs failed. Then the velocities decreased quickly to form a "dead zone" in front of the fuseplug farthest to the right. Velocities at the next upstream radial station (locations No. 10-14) decreased in magnitude from left to right across the channel as would be expected from the flow through the failed sections on the left. The velocities at the next upstream radial station (locations No. 15-19), showed the effect of the curved approach channel. These velocities were lower than the velocities at the previous station on the left side (where the plugs had failed) but increased across the channel to the right where the velocities were greater than the most immediate upstream section (figs. 11b, c, and d). With all fuseplugs failed, the velocities increased across the approach channel from left to right, and from the reservoir towards the control section (fig. 11e). These velocities were used in empirical equations to predict the possibility of approach channel erosion. The result of these analyses was that significant erosion should not occur.

Velocities on the upstream slope of the fuseplugs. — To determine the effect of flow through failed fuseplugs on adjacent embankments, velocities were measured on the upstream face of an adjacent fuseplug when the fuseplug section to the left failed. Velocities were measured with a Nixon propeller meter at the surface and normal to the 2:1 upstream slope of the fuseplugs. All the splitter walls were constructed alike to match the geometry of the fourth fuseplug section. Therefore, the splitter walls, between the first and second, and second and third fuseplugs, extended horizontally upstream from the fuseplug embankment slopes by 8 and 4 feet, respectively.

Velocities were measured in 12 locations normal to and along the fuseplug upstream slopes: four at the toe, four 22 feet up the slope, and four 42.4 feet up the slope. The measurements were taken within the first 20.6 feet to the right of each splitter wall.

Contours of the velocities on the face of the second through fourth fuseplugs are shown on figures 12a through 12c. The actual velocity magnitudes are shown at the locations where they were measured. The velocities were used to determine the size of riprap needed to protect the upstream face of the fuseplugs to prevent premature failure of the plugs. To be able to use reasonable, about 2-foot-, size riprap, the velocities had to be less than or equal to 13 ft/s. The velocities on the third plug from the left were in this range near the wall and toe. These velocities were considered acceptable by the designers because the 4-foot wall extension would minimize erosion damage. The velocities on the face of the fourth plug exceeded the criteria. Therefore, the splitter wall, which had been flush with the embankment section, was modified to extend upstream by 5 feet normal to the slope. This modification greatly reduced the velocities on the face of the embankment, ensuring the stability of the riprap protection.

Velocities at the end of the excavated fuseplug exit channel. — To determine the erosion potential, velocities were measured at the downstream end of the 5-percent slope of the fuseplug exit channel where the cut daylights into the existing topography. The velocities were measured, with an OTT meter at about one-half the 19.5-foot flow depth, at the maximum flow rate with all fuseplugs failed. The velocities averaged about 49 ft/s all along the edge of the cut.

Erosion of the rock surface is expected to occur under these velocities; however, erosion is not expected to headcut far enough upstream to undermine the concrete control sill.

Velocities adjacent to the footing of the 365-kilovolt transmission tower in the reservoir. — The footing of the existing transmission tower is located on a knoll in the reservoir 40 feet to the left of the fuseplug centerline and 750 feet upstream from the fuseplug axis. The base of the tower is submerged by 1 foot at reservoir El. 1825. Velocities adjacent to the tower footing were measured to determine if the potential to erode the material near the footing existed. Measurements were taken on four radial lines, to about 80 feet out into the reservoir, ranging from parallel to perpendicular to the fuseplug axis. Velocities were measured with a small Nixon propeller meter. The maximum velocities, up to 9.9 ft/s, were measured on the line at 45° to the tower. The velocities are not considered great enough to produce sufficient erosion to undermine the transmission tower footing.

Disposal of excavated material. — Excavation to the fuseplug sill elevation of 1780 will produce approximately 225,000 yd^3 of material for disposal. To reduce costs, the most likely places to dispose of the material are in the reservoir or near the excavated control sill and exit channel.

The material sizes are such that disposal in the reservoir would be an acceptable alternative from an environmental standpoint. Depositing the material in the reservoir immediately upstream from the approach channel to El. 1780 was investigated in the model. The result was unsatisfactory, because the increased length and friction in the curved approach channel produced an undulating water surface and reduced the capacity of the fuseplug section. The material was then removed from the approach channel for the remainder of the testing.

Material was then placed above the cut slope to the left side of the approach channel in the small drainage area immediately upstream from the fuseplug spillway axis. Placing material in this area improved the flow conditions by eliminating a small contraction that had previously been produced by the local topography. This material would be under water only during flood stage. This location was hydraulically acceptable for material disposal and the material remained in the location for the remainder of the testing.

Investigations continued by placing material in a dike from the downstream end of the right side of the fuseplug section across an existing drainage channel. Failure of any of the fuseplugs would subject this channel to erosion from a small portion of the flow from the fuseplug spillway failure. The toe of the dike was oriented 26.5° back from the toe of the cut slope on the right side of the fuseplug channel and extended about 750 feet to the largest knoll downstream. The dike would need to be about 25 feet at the upstream end and could taper down to about 13 feet at the downstream end next to the knoll. The dike could be blended into the existing topography as needed, provided that the toe of the dike does not extend into the flow path.

Final Modeled Fuseplug Design

The initial fuseplug was designed based upon a conservative discharge coefficient, recommended by the VE team, and maximum reservoir water surface El. 1825. The maximum water surface under which the dam is stable is El. 1820.80. Therefore, the final modeled fuseplug design was determined using data from the initial model results and flood routings up to the new maximum water surface restriction. The final modeled fuseplug design has a total bottom width of 357 feet with three fuseplug sections on the control sill at El. 1780. The bottom widths of the fuseplugs were 150, 110, and 70 feet from left to right, respectively. The side slopes of the approach channel and control section remained at 3/4:1. The top of each fuseplug embankment and each pilot channel are separated by a 3-foot increment. Each pilot channel is 1 foot deep. The three fuseplugs are separated by two splitter walls, 0.2:1 batter, each with a bottom width of 13.5 feet and a height of 30 feet.

Discharge rating and flow conditions. — The final discharge rating curve for the auxiliary fuseplug spillway is shown on figure 15. The maximum discharge when all three fuseplugs have failed is 265,250 ft^3/s at reservoir El. 1820.80. The maximum discharge when the first fuseplug has failed is 55,550 ft^3/s at reservoir El. 1806. The maximum discharge when the first and second fuseplugs have failed is 116,500 ft^3/s at reservoir El. 1809. The discharge coefficient, based on the bottom width of each fuseplug section, varies from about 2.50 at low heads to 3.08 at higher heads (fig. 16).

Flow conditions through the approach channel and the control section remained almost identical to those of the earlier design. Critical depth occurred on the control sill for all flow rates. The large contraction on the left side of each splitter wall remained until the fuseplug section was fully open. The flow accelerated along the curved right wall of the approach channel. The losses through the approach channel produced drawdown in the channel that again increased the reservoir elevation before breaching of the second and third fuseplugs began. With the first fuseplug failed, the second fuseplug will begin breaching at reservoir El. 1806.46 (+0.46). With the first two fuseplugs failed, the third fuseplug will begin breaching at El. 1811.24 (+2.24 ft). The pilot channel on the third fuseplug should be moved away from the right side, toward the center, or left to assure breaching as soon as possible. The flow conditions associated with failure of each fuseplug section are shown on figure 25.

Flow profiles, depths, and velocities. — The flow depths were determined by recording water surface profiles through each section for the maximum flow condition through each section before failure of the next fuseplug and eventually for the fully open section. The profiles were developed for a control section with 3/4:1 side slopes as in the initial design. These profiles are shown on figure 19 with the vertical depth computed at the fuseplug axis given in table 2. The depths through the first section increased slightly because the second fuseplug pilot channel is 1 foot higher than the second fuseplug pilot channel in the initial design. Depths through the second fuseplug section closely match those recorded for the third fuseplug in the initial design because both depend upon the pilot channel of the last fuseplug, which was at the same elevation in both designs. The water depths through the fully open section are slightly less because of the decrease in the maximum reservoir water surface from El. 1825 to El. 1820.8.

Table 2. — Vertical depths at the fuseplug axis for the side walls and piers.

No. of failed plugs	Reservoir elevation (ft)	Channel No. 1		Channel No. 2		Channel No. 3	
		Left side slope	Right wall	Left wall	Right wall	Left wall	Right side slope
1	1806	20.86	6.11	–	–	–	–
2	1809	23.47	14.78	25.58	10.26	–	–
3	1820.8	27.98	Over wall	Over wall	24.90	28.37	27.87

Velocities in the approach channel, along the fuseplug embankments, and through and downstream from the control sill, were measured again. Velocity magnitudes in the approach channel and in the centerline of failed fuseplug sections are shown on figure 17. These bar graphs show the velocities associated with the discharge distributions when the number of fuseplugs was reduced from four to three. The magnitudes (table 3) show similar trends to those of the initial design and were again used in empirical equations for determination of possible erosion. The result of these analyses was that no significant erosion would occur.

The velocities normal to the face of the second and third fuseplugs were measured to check the effect of the different flow quantities through failed sections on the erosion potential of adjacent embankments (table 4). These measurements show (compare figs. 11b, 12a, and 17b, 18a) an increase in the velocities on the second fuseplug face because of the greater amount of flow through the first open section, but the velocities were acceptable. The velocities on the face of the last fuseplug were less than those on the third fuseplug section from the initial design (compare figs. 11c, 12b, and 17c, 18b) because less total flow was discharged through the open sections. The contour plots of the velocities, with the actual values shown at the measurement locations, are shown on figure 18.

Table 3. — Approach and through channel velocities for the final fuseplug spillway design.

Location No.*	Velocity (ft/s)			Distance across channel from left to right (ft)
	One plug failed	Two plugs failed	All plugs failed	
A	25.4	25.2	27.24	75 to centerline of channel No. 1
B	0	28.11	28.15	218.5 to centerline of channel No. 2
C	0	0	28.03	322 to centerline of channel No. 3
1	12.45	17.33	23.36	10
3	12.12	17.12	23.53	75
5	10.58	17.24	25.7	156.75
6	0	16.83	25.9	218.5
7	0	13.53	25.74	280.25
8	0	8.66	27.15	322
9	0	0	26.7	347
10	5.75	11.45	16.74	10
11	6.17	11.04	18.45	75
12	5.75	12.12	20.82	163.5
13	5	12.08	23.53	266
14	2.29	9.96	26.23	347
15	3.42	7.29	12.79	10
16	4.17	9.62	15.54	75
17	5.17	11.45	18.08	163.5
18	5.12	11.87	21.32	266
19	5.12	12.37	21.7	347
20	4.63	11.33	20.49	347

* Refer to figure 17a.

Table 4. — Velocities on the surface of and normal to the face of the fuseplugs for the final design.

Location No.	Velocity normal to face of plug (ft/s)		Location to right of wall (ft)	Distance up 2:1 slope from toe (ft)
	No. 2	No. 3		
2	7.95	2.87	Adjacent to wall	39
3	9	3.53	6.88	39
4	8.1	2.62	13.75	39
5	8	1.43	20.63	39
7	10.35	6.76	Adjacent to wall	20
8	11.02	7.89	6.88	20
9	9.37	6.37	13.75	20
10	8.03	4.88	20.63	20
12	8.39	6.26	Adjacent to wall	Toe
13	9.69	8.71	6.88	Toe
14	8.57	7.57	13.75	Toe
15	8.09	5.48	20.63	Toe

Velocities were measured in the downstream channel where the excavation daylights to the existing topography under the maximum discharge at reservoir El. 1820.80. The average velocity, 46 ft/s, was slightly less than in the initial design because of the lower discharge at reservoir El. 1820.80.

Velocities in the reservoir adjacent to the base of the transmission tower were insignificant when measured with the initial design at reservoir at El. 1825. The base of the transmission tower is at El. 1824, out of the water under the final maximum reservoir water surface El. 1820.80. Therefore, any potential for erosion at the lower reservoir water surface would be even more remote.

***Final fuseplug section design.* —** As the design became finalized, the cut slopes in the control section, thus the approach channel, were steepened to 1/2:1 from the 3/4:1 slopes that were modeled. This change reduced the cross sectional area of the control section slightly. Computations were made, using the depths from the previous model tests, to determine the additional bottom width of each section that would offset the lost area in the side slopes. The maximum water surface will remain at El. 1820.80, and the additional bottom width will pass the PMF. Figure 26 and the following table describe the final geometry of the fuseplugs:

Table 5. — Final fuseplug geometry.

Fuseplug No.	Fuseplug width at bottom (ft)	Fuseplug El. (ft)	Pilot channel El. (ft)
1 (left looking downstream)	155.25	1804	1803
2 (middle)	111	1807	1806
3 (right looking downstream)	74.5	1810	1809

The total bottom width of the section is 367.75 feet with 1/2:1 side slopes. The batter on the two splitter walls remained at 0.2:1 for the 30-foot height, accounting for 27 feet of the toal width.

Sectional Model of Approach Channel Erosion

A 1:43.2 scale sectional model was constructed in a 3-foot-wide flume to simulate erosion of rock blocks along the floor of the excavated fuseplug approach channel. The sectional model represented a 129.6-foot width of the approach channel with a 14.4-foot-wide section of individual blocks placed over a gravel bed along the centerline. In cross section, the sectional model included a 3:1 upstream transition slope from the floor of the flume to a modeled 54-foot-long flat section with a 5-percent downstream sloping section representing the exit channel (fig. 27). The 1/2-inch aluminum blocks represented 1.8-foot-size prototype rock blocks with the same specific gravity as granite. The block size was determined from the available field drill hole information. The blocks were placed three layers deep over the pea gravel bedding, which allowed water to saturate the bedding and produce uplift pressure. The blocks were randomly placed to varying degrees of tightness across the channel, forming a fairly smooth transition with the adjacent surfaces.

The flow rate was slowly raised over the section until the unit discharge represented that of the PMF discharge. The blocks were stable until the discharge reached about one-half the PMF, at which time blocks eroded from the downstream end of the horizontal section. As the discharge increased, about 18 rows of blocks eroded from the top layer of blocks only. Erosion of the blocks continued upstream until encountering the first row of tight blocks (fig. 28). The erosion mechanism seemed to be primarily uplift with local disturbances greatly increasing the erosion. This test was successfully duplicated after the blocks were replaced to their original condition. This information gave a further qualitative look at the expected erosion pattern in the fuseplug approach channel. The tests were interesting in that they showed that tight rock masses should not erode or would deter the continuation of initiated erosion. The tests also verified the erosion predicted by the tractive force analysis (USBR, 1993). They also showed that the erosion should not progress to significant depths, thus the small cutoff trench proposed at the upstream end of the concrete sill should be adequate.

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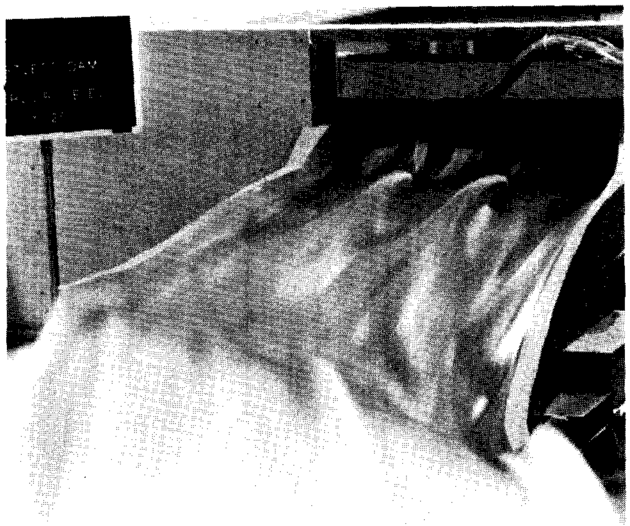


Figure 1. — Overall view of the Bartlett Dam 1:60 scale service spillway model.

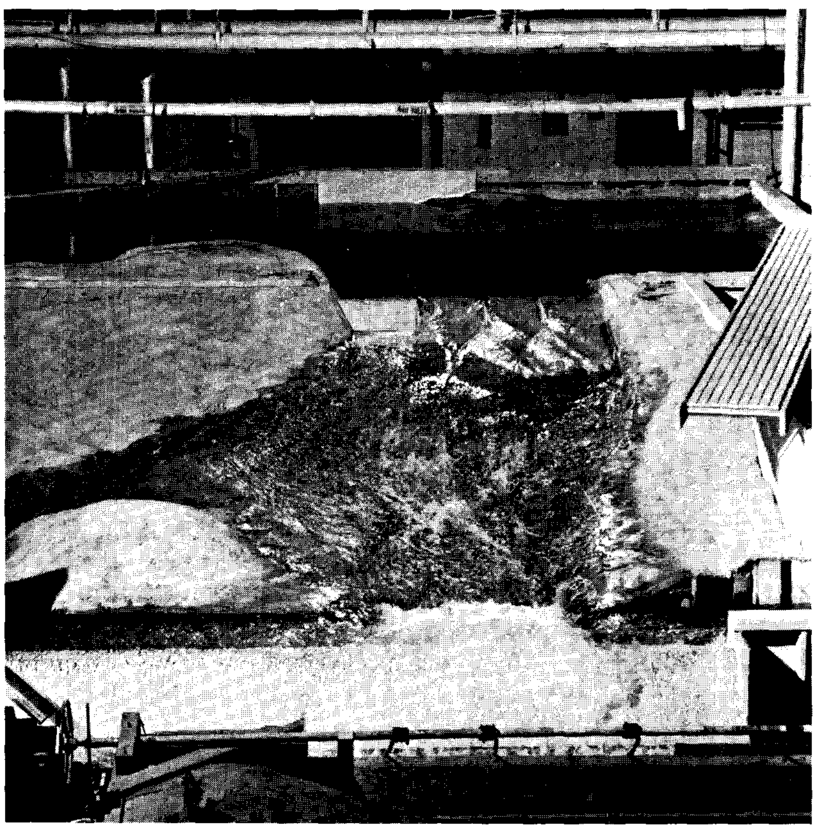


Figure 2. — Overall view of the Bartlett Dam 1:55 scale initial auxiliary fuseplug model design.

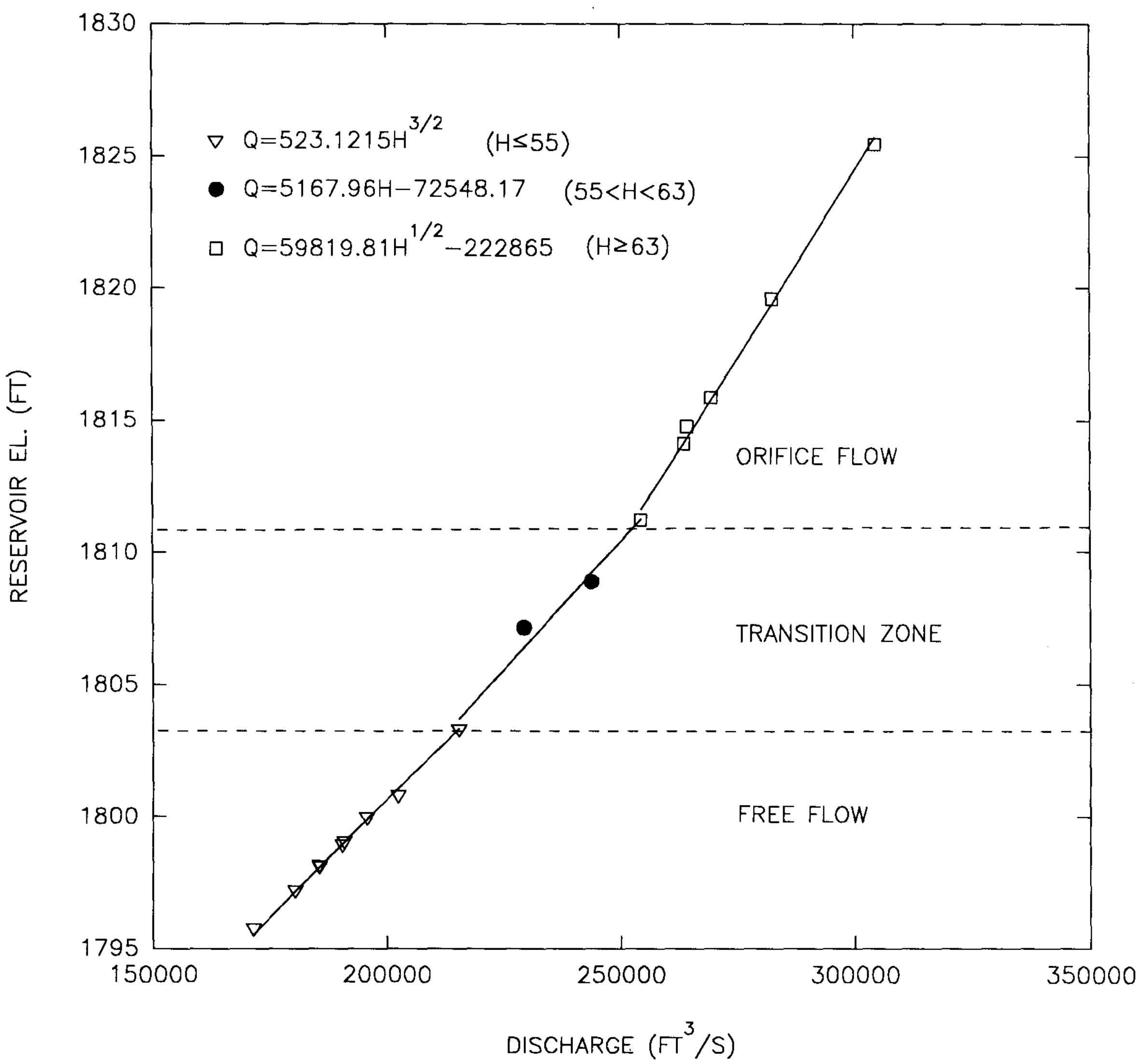


Figure 3. — Service spillway discharge rating curve for ungated flows above 175,000 ft^3/s.

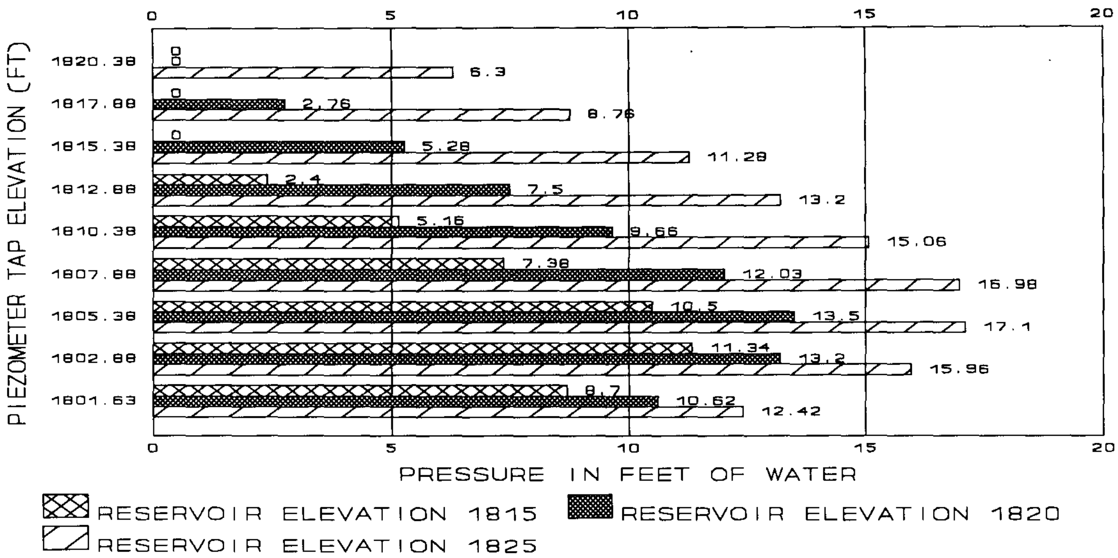


Figure 4. — Pressure profiles on the upstream vertical face of the bridge spanning the service spillway.

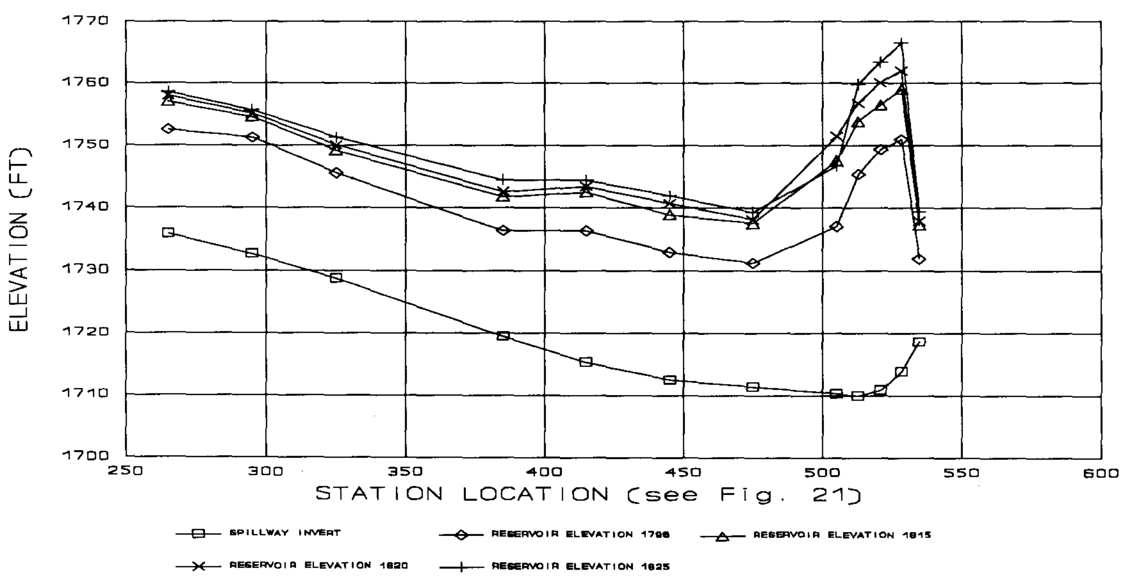


Figure 5. — Pressure profiles along the centerline of the spillway chute invert from Sta. 2+50 through the flip bucket.

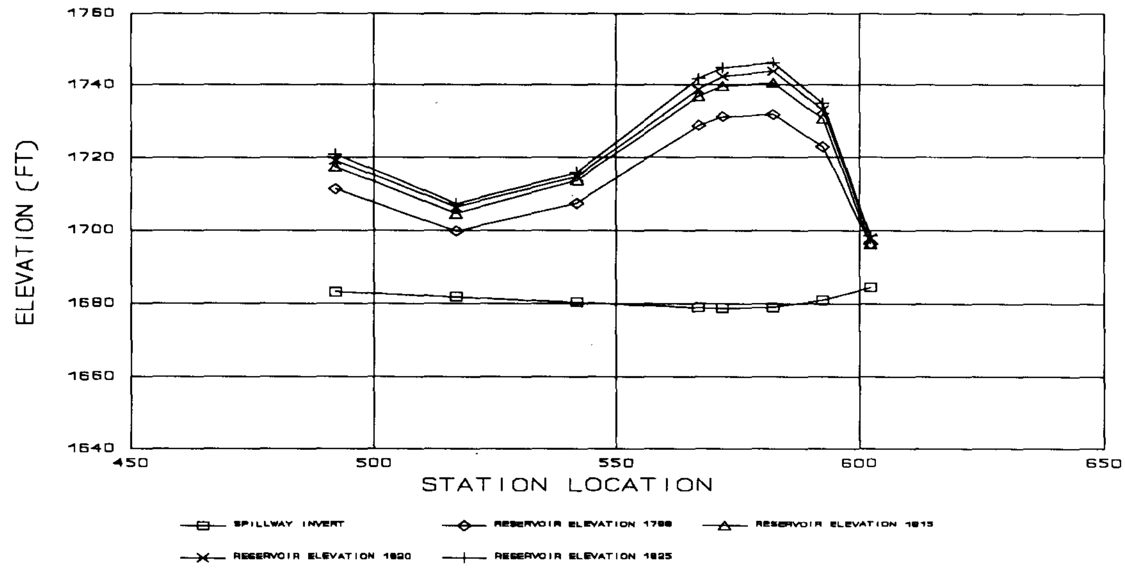


Figure 6a. — Pressure profiles along the invert of the chute at the base of the left wall from Sta. 4+91.97 through the flip bucket.

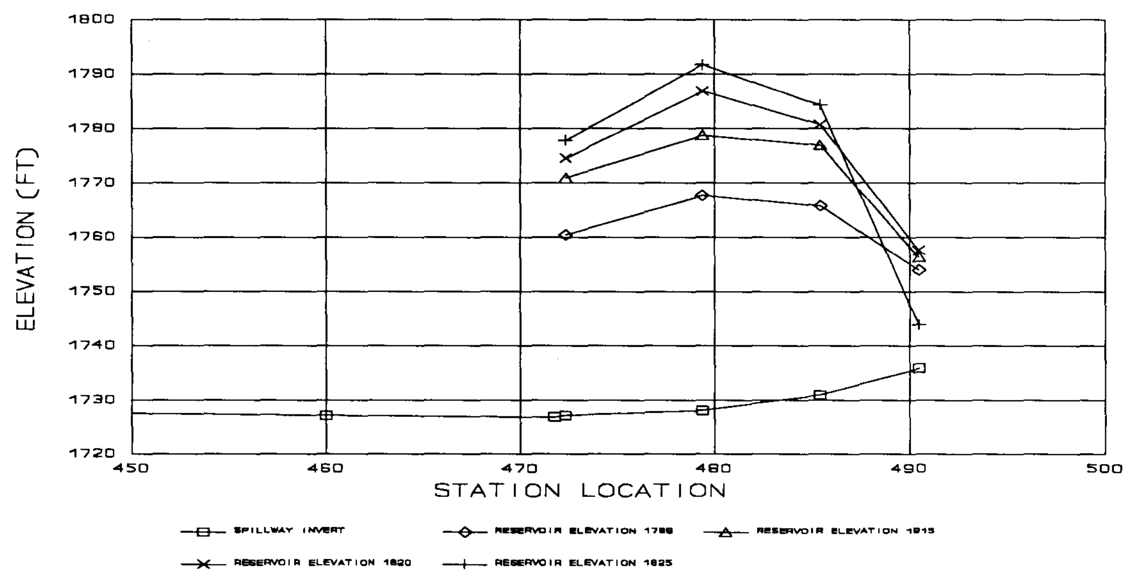


Figure 6b. — Pressure profiles along the invert of the chute at the base of the right wall through the flip bucket.

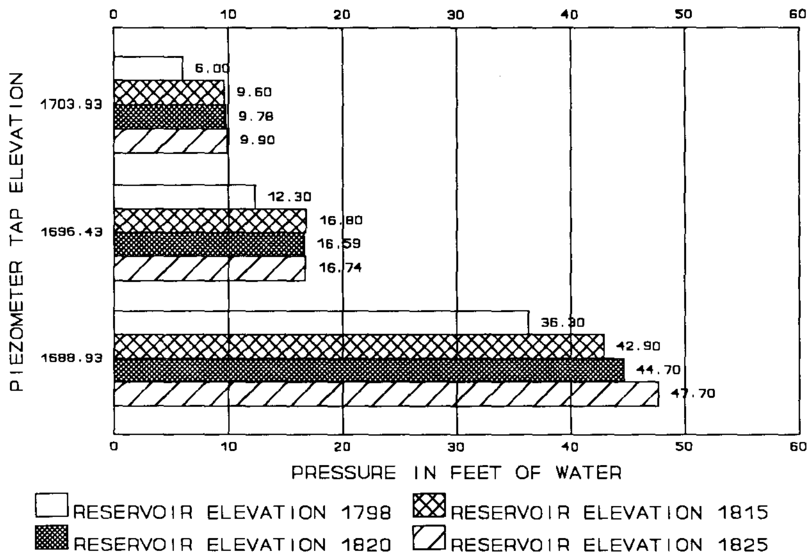


Figure 7. — Pressure magnitudes on the left wall of the flip bucket at Sta. 5+93.5.

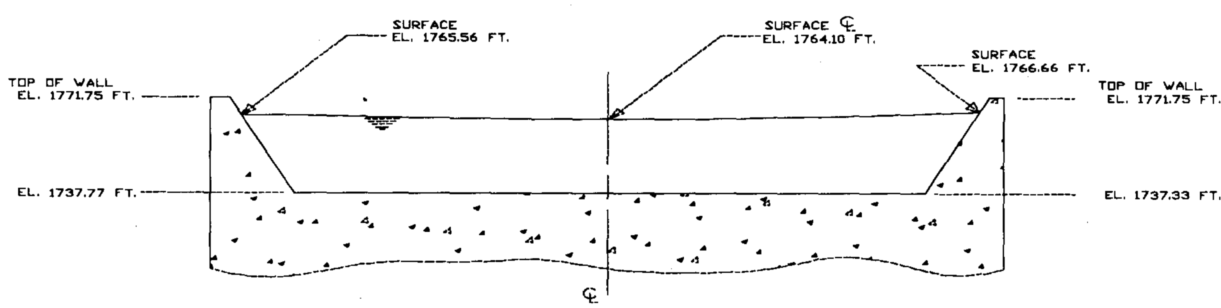


Figure 8a. — Cross section of flow depth on the superelevated spillway chute at Sta. 2+50 shown vertical to the invert on the chute centerline.

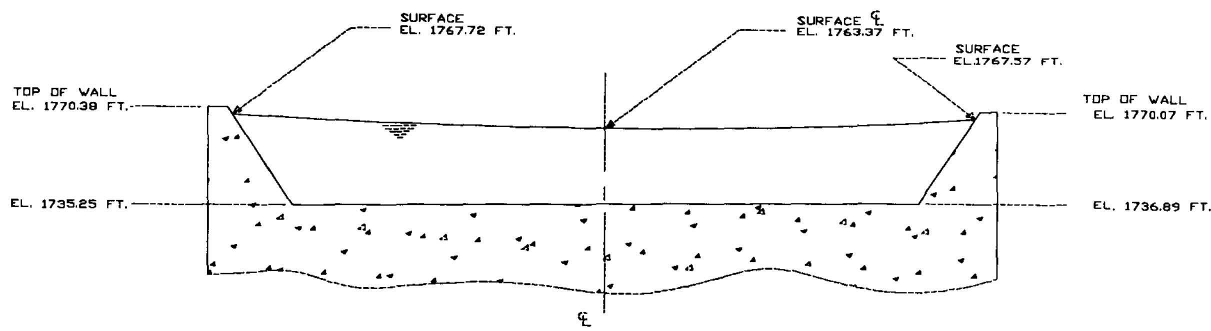


Figure 8b. — Cross section of flow depth on the superelevated spillway chute at Sta. 2+65 shown vertical to the invert on the chute centerline.

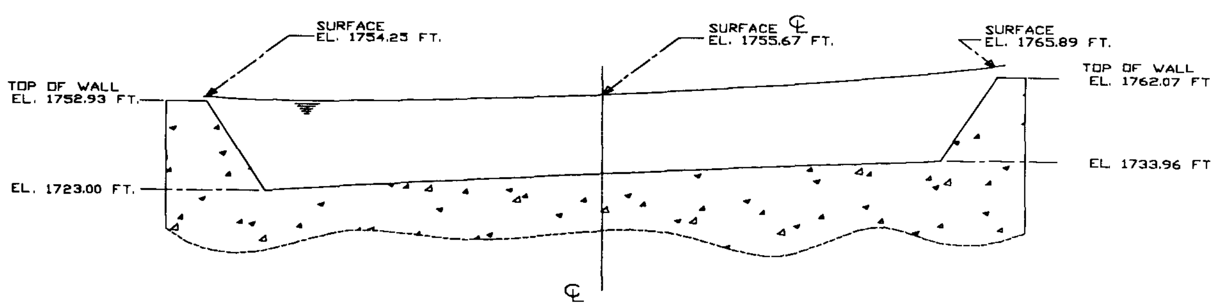


Figure 8c. — Cross section of flow depth on the superelevated spillway chute at Sta. 3+25 shown vertical to the invert on the chute centerline.

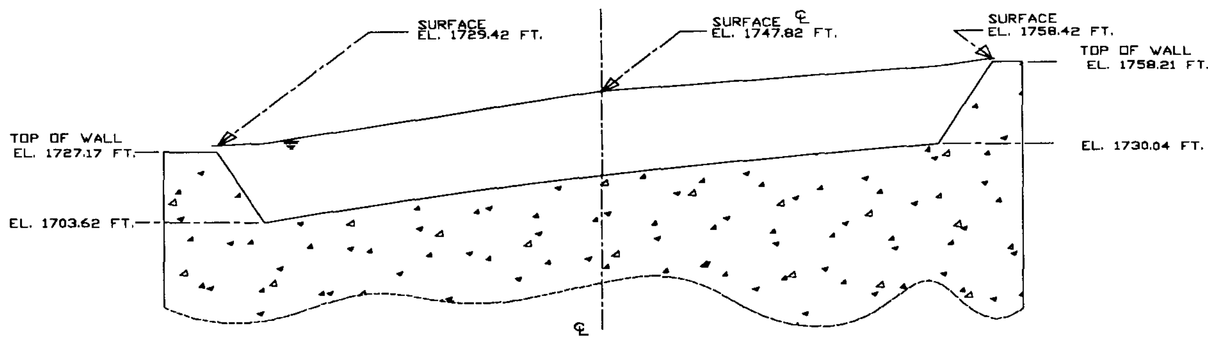


Figure 8d. — Cross section of flow depth on the superelevated spillway chute at Sta. 3+85 shown vertical to the invert on the chute centerline.

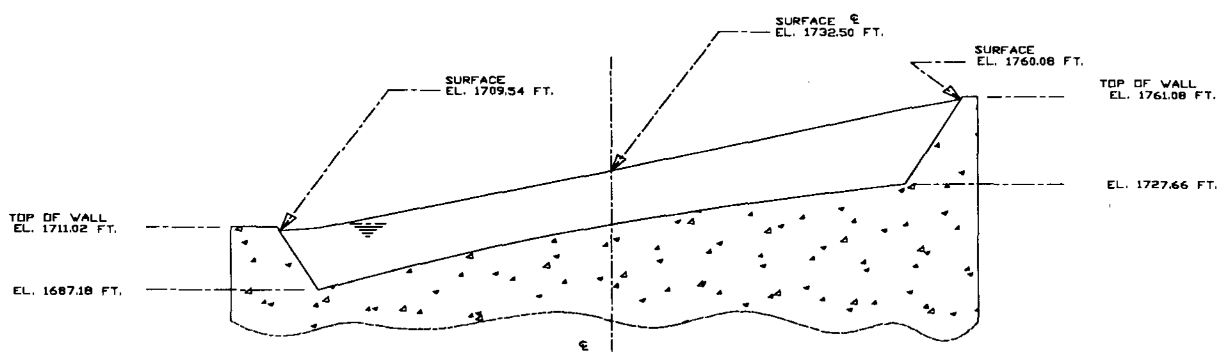


Figure 8e. — Cross section of flow depth on the superelevated spillway chute at Sta. 4+45 shown vertical to the invert on the chute centerline.

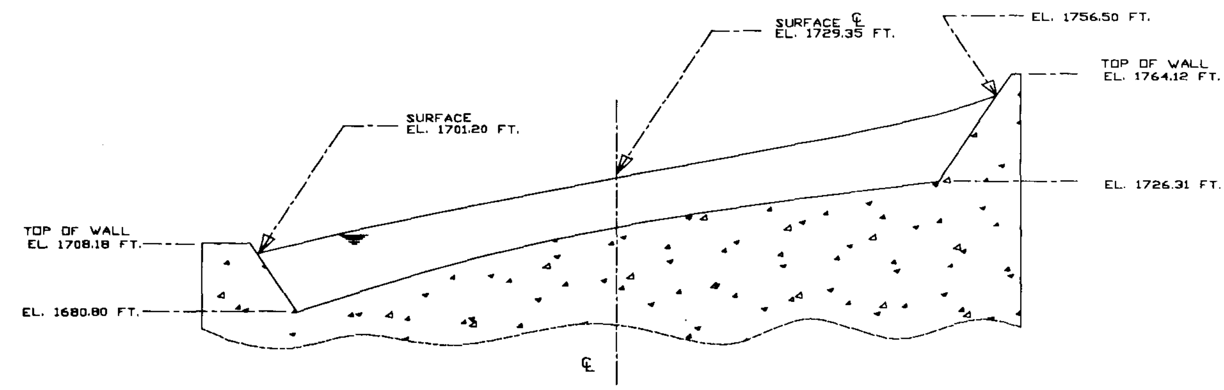


Figure 8f. — Cross section of flow depth on the superelevated spillway chute at Sta. 4+90 shown vertical to the invert on the chute centerline.

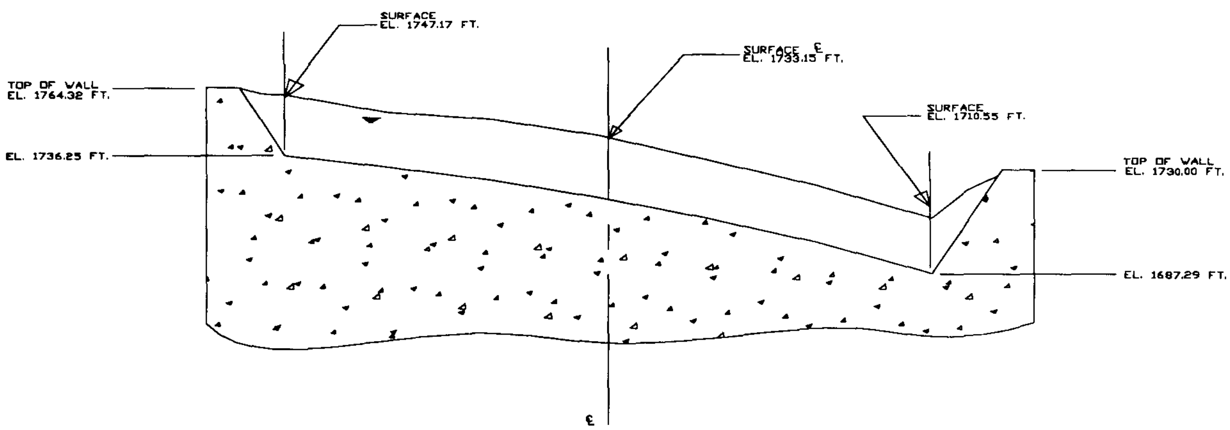


Figure 8g. — Cross section of flow depth on the superelevated spillway chute at the end of the flip bucket shown vertical to the invert on the chute centerline.

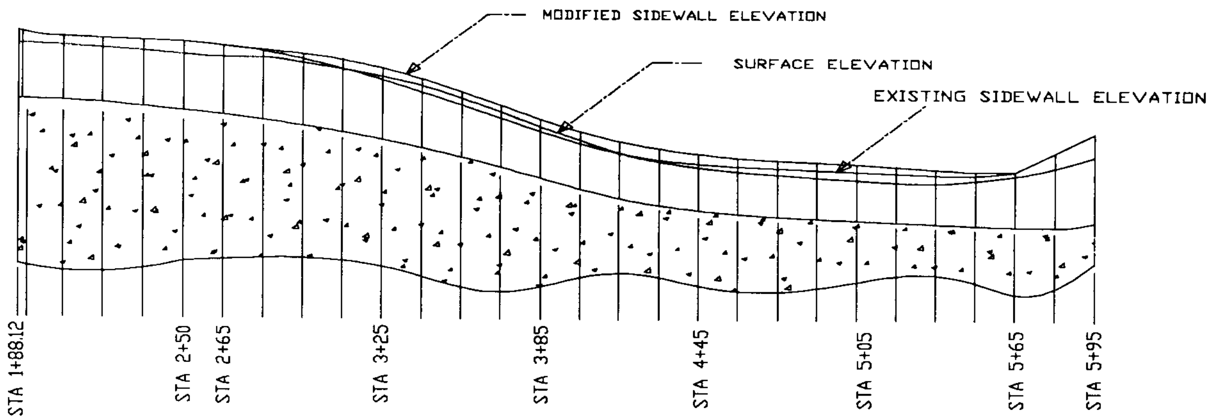


Figure 9a. — Water surface profile along the left chute wall showing locations and amounts of flow overtopping the existing chute wall.

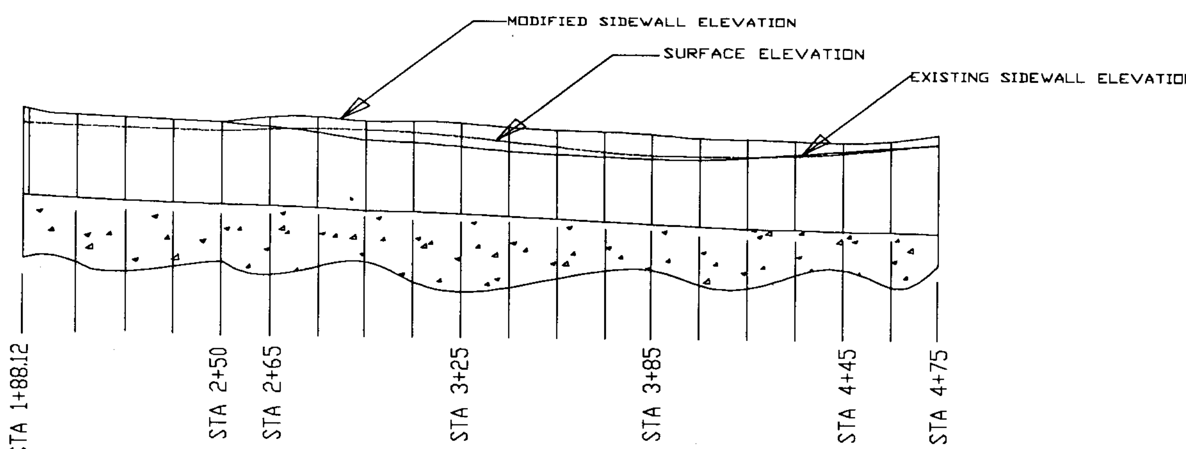


Figure 9b. — Water surface profile along the right chute wall showing locations and amounts of flow overtopping the existing chute wall.

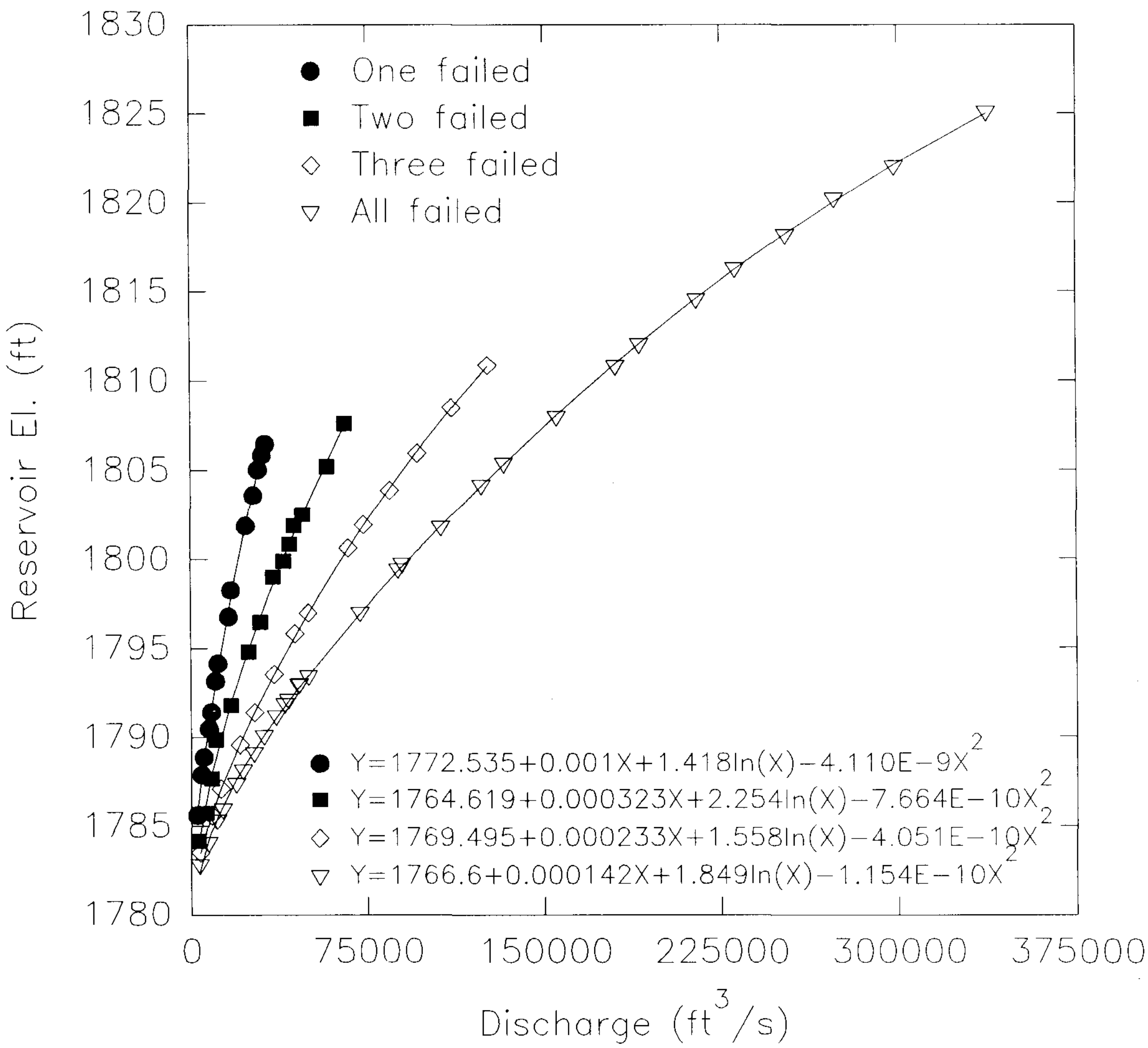


Figure 10. — Discharge rating for initial fuseplug sections.

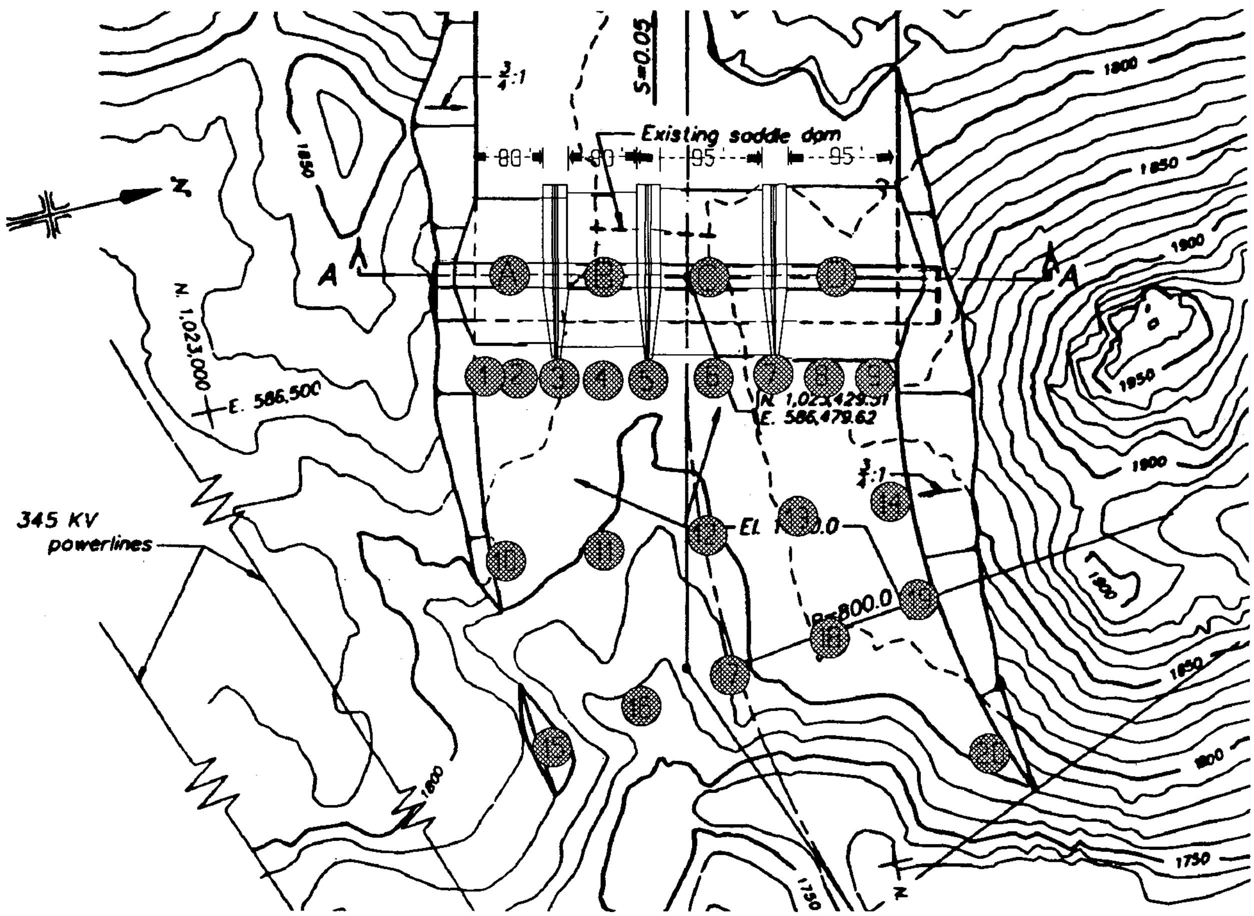


Figure 11a. — Location map for velocities measured in the fuseplug approach channel for the initial design.

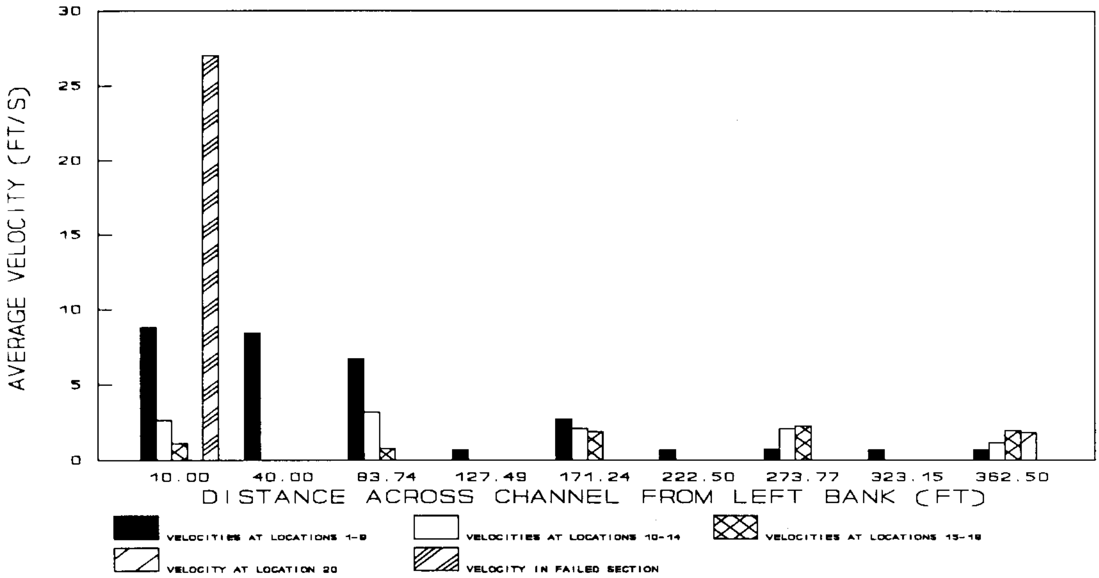


Figure 11b. — Velocity magnitudes in the approach channel and through the first failed section for the initial design.

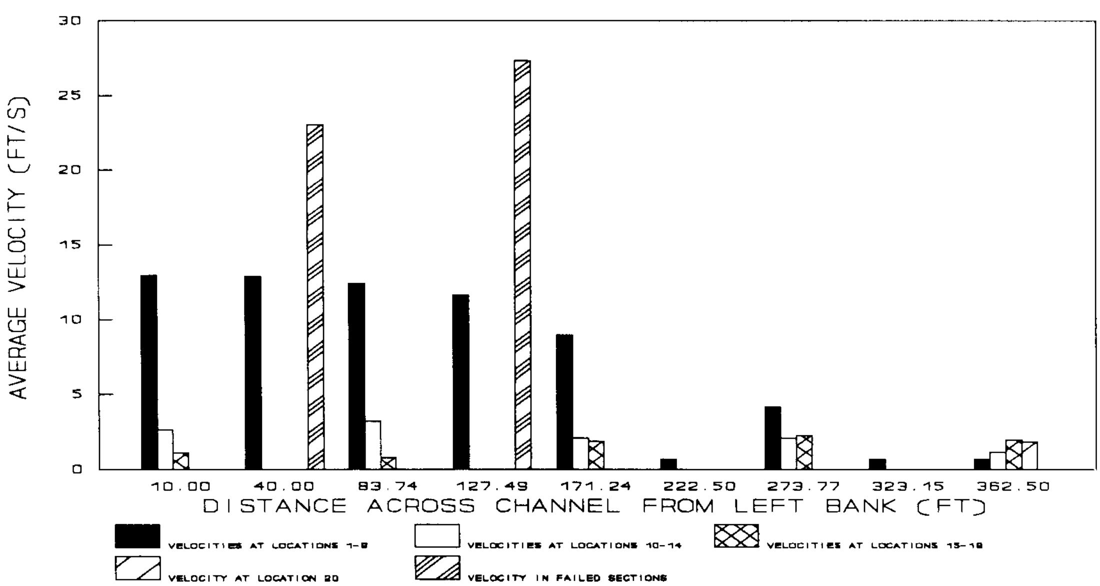


Figure 11c. — Velocity magnitudes in the approach channel and through the first two failed sections for the initial section.

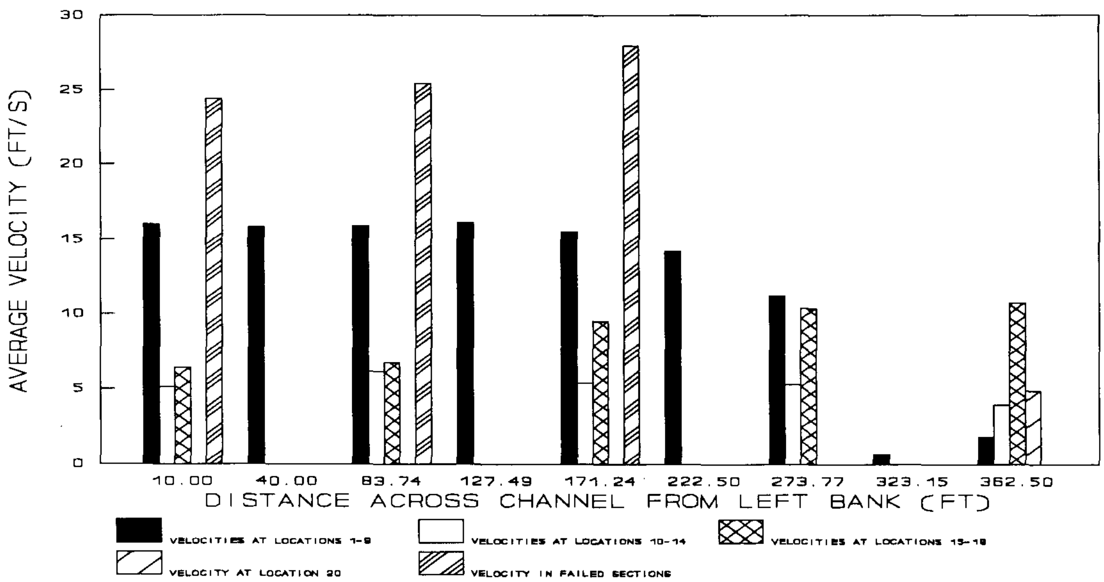


Figure 11d. — Velocity magnitudes in the approach channel and through the first three failed sections for the initial design.

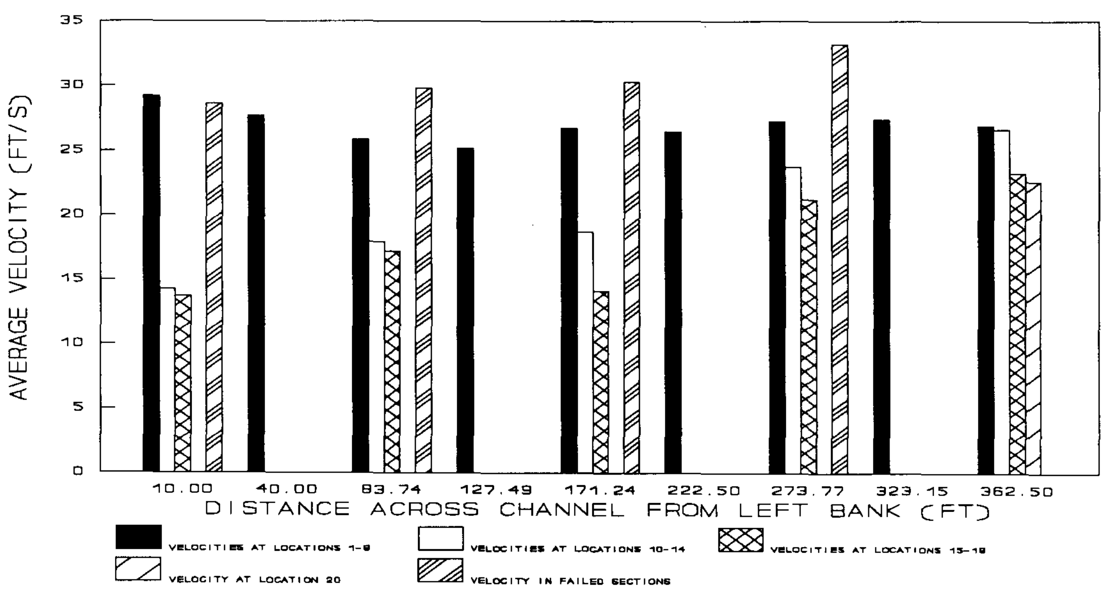


Figure 11e. — Velocity magnitudes in the approach channel and through the entirely failed section for the initial design.

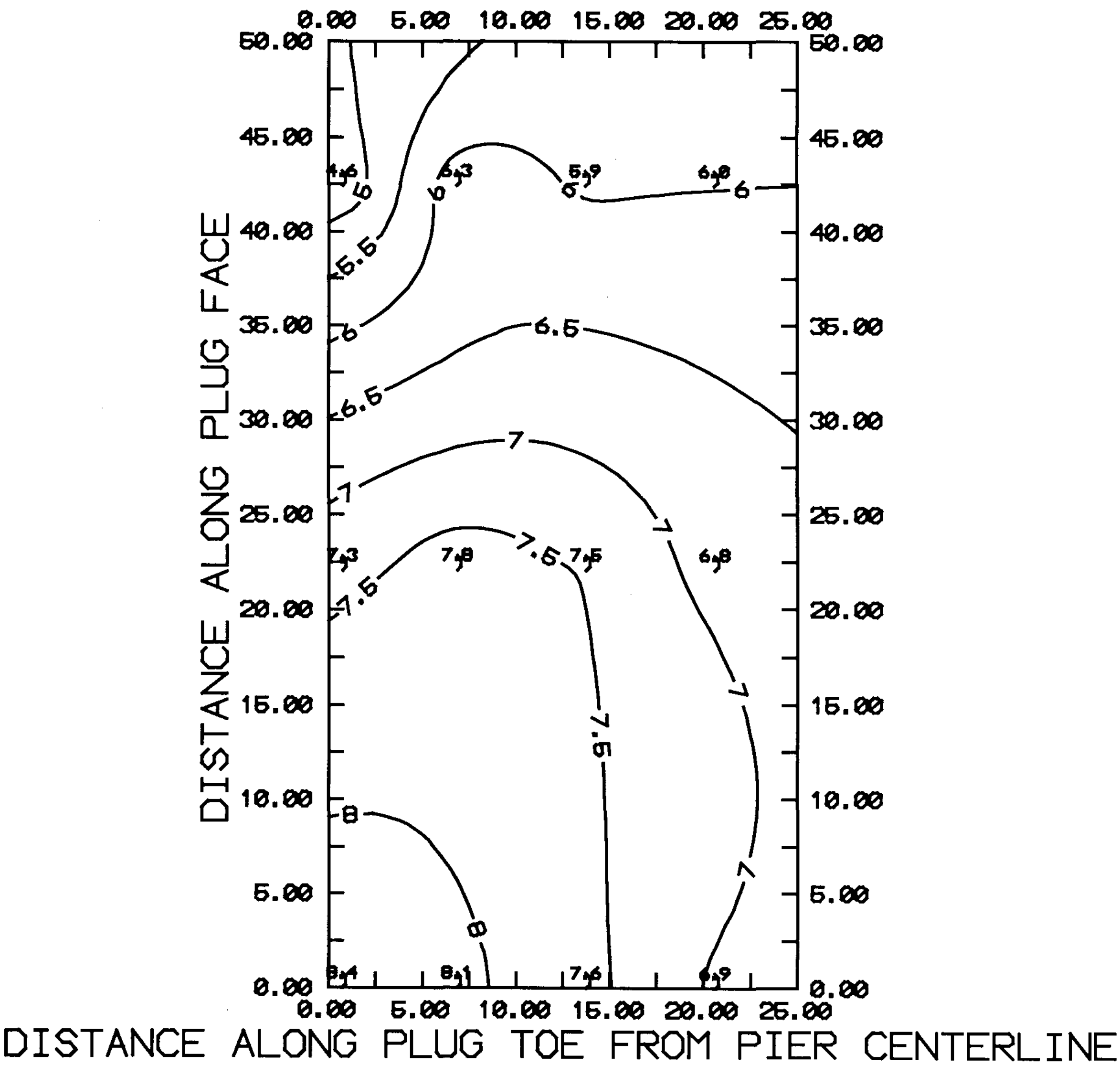


Figure 12a. — Velocity contours on the face of and normal to the second fuseplug embankment for the initial design.

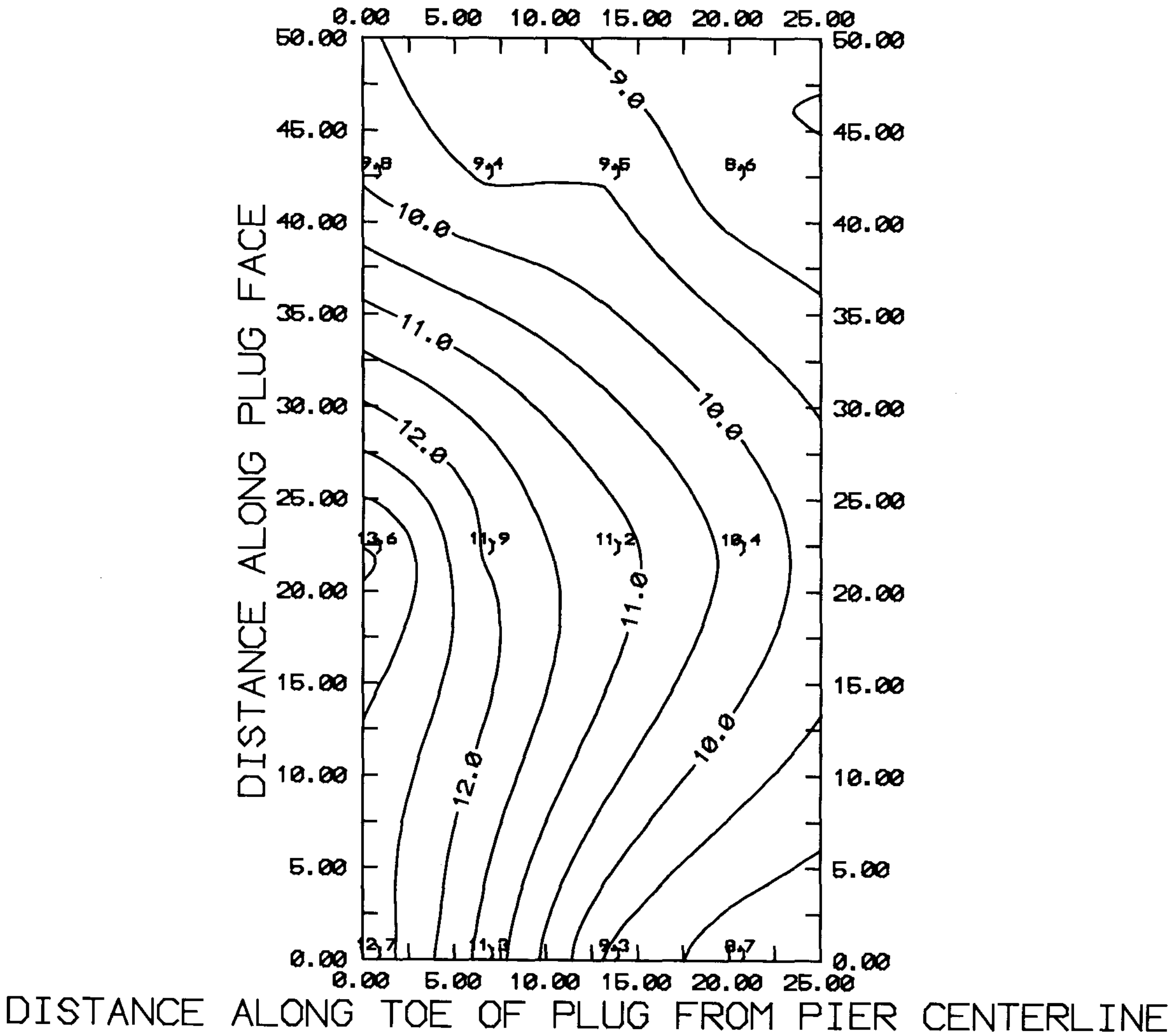


Figure 12b. — Velocity contours on the face of and normal to the third fuseplug embankment for the initial design.

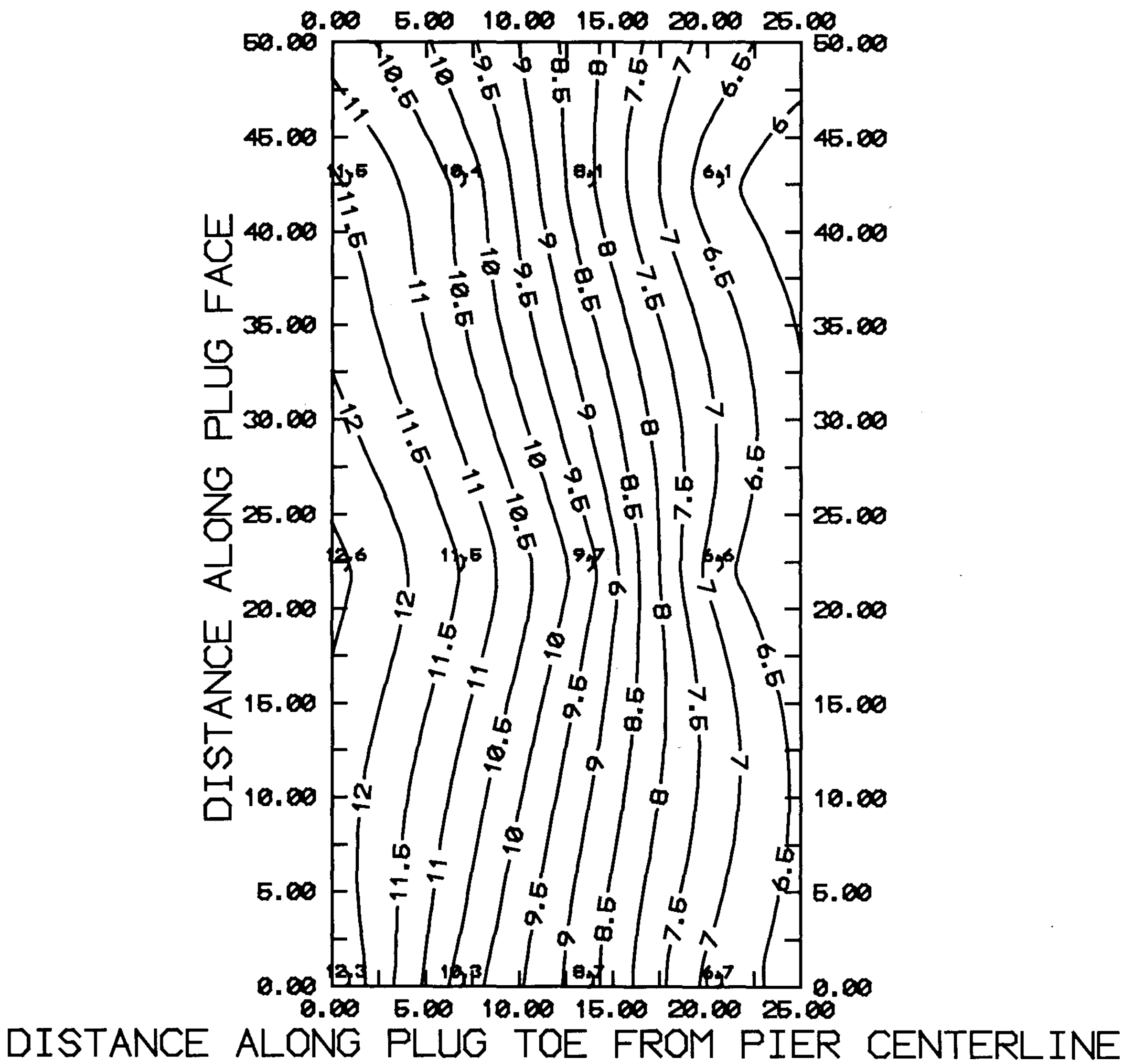


Figure 12c. — Velocity contours on the face of and normal to the fourth fuseplug embankment for the initial design.

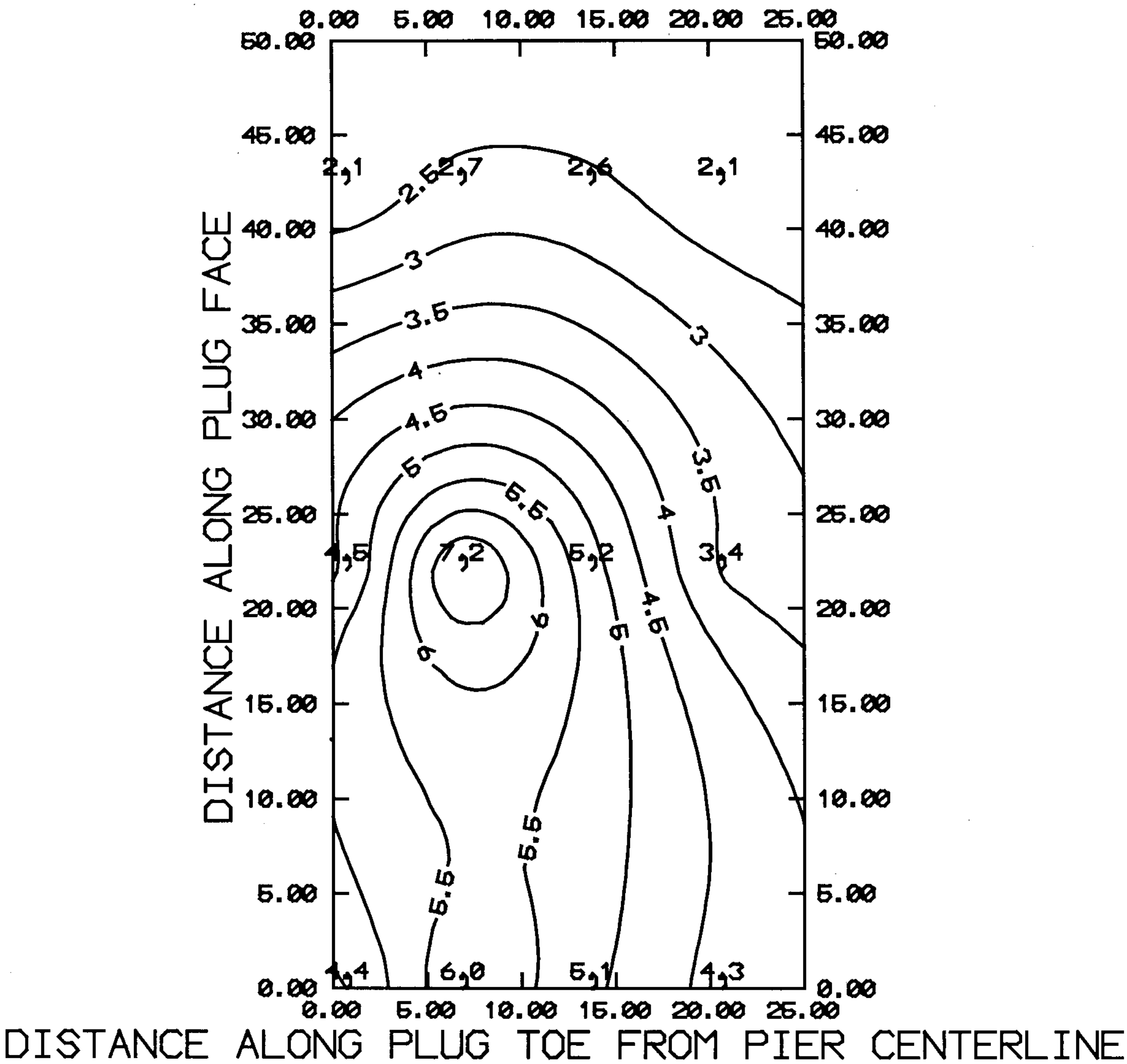


Figure 12d. — Velocity contours on the face of and normal to the fourth fuseplug embankment for the initial design with the extended splitter wall.

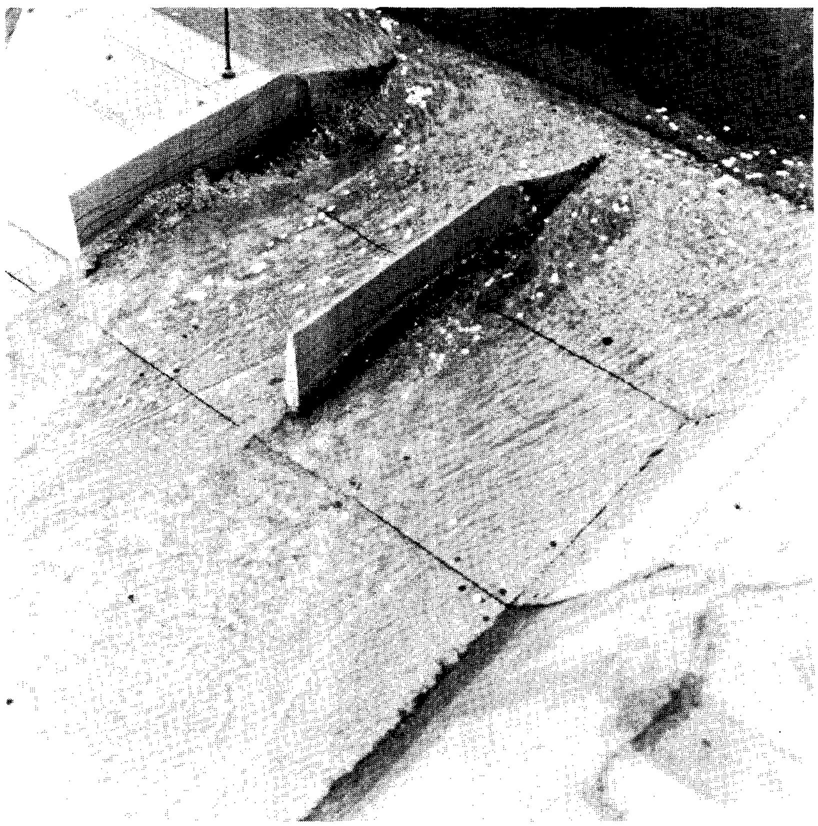


Figure 13. — Typical flow condition through failed fuseplug sections showing the contraction along the splitter wall.

Figure 14. — Proposed downstream dike location (shown by sand bags).

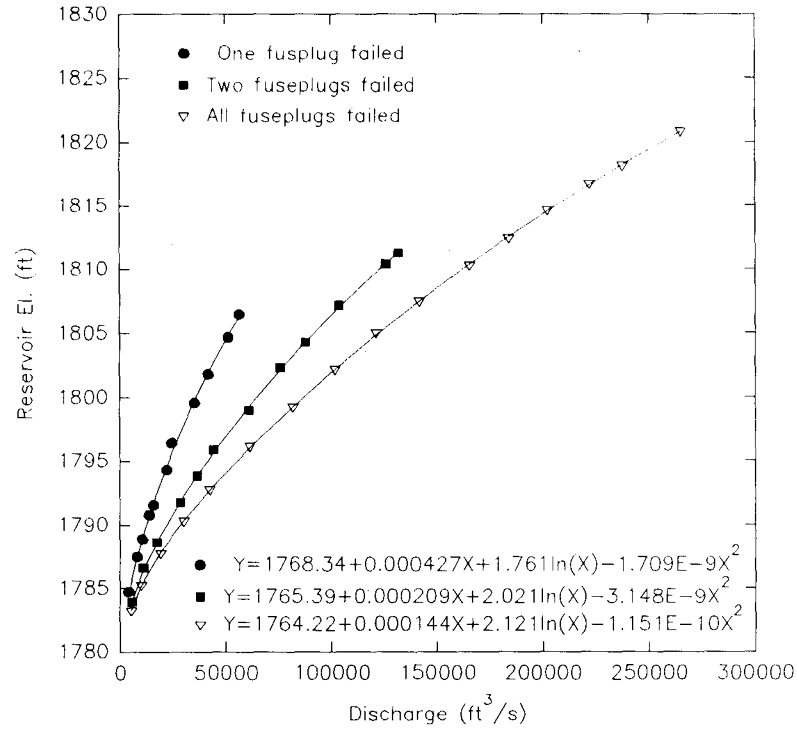


Figure 15. — Discharge for the final fuseplug sections.

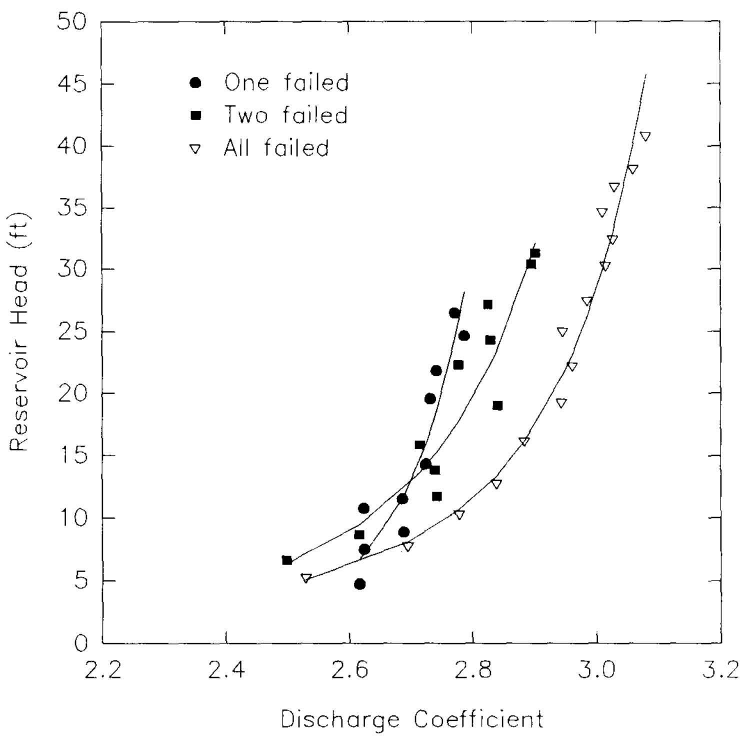


Figure 16. — Discharge coefficient curves for the final fuseplug sections.

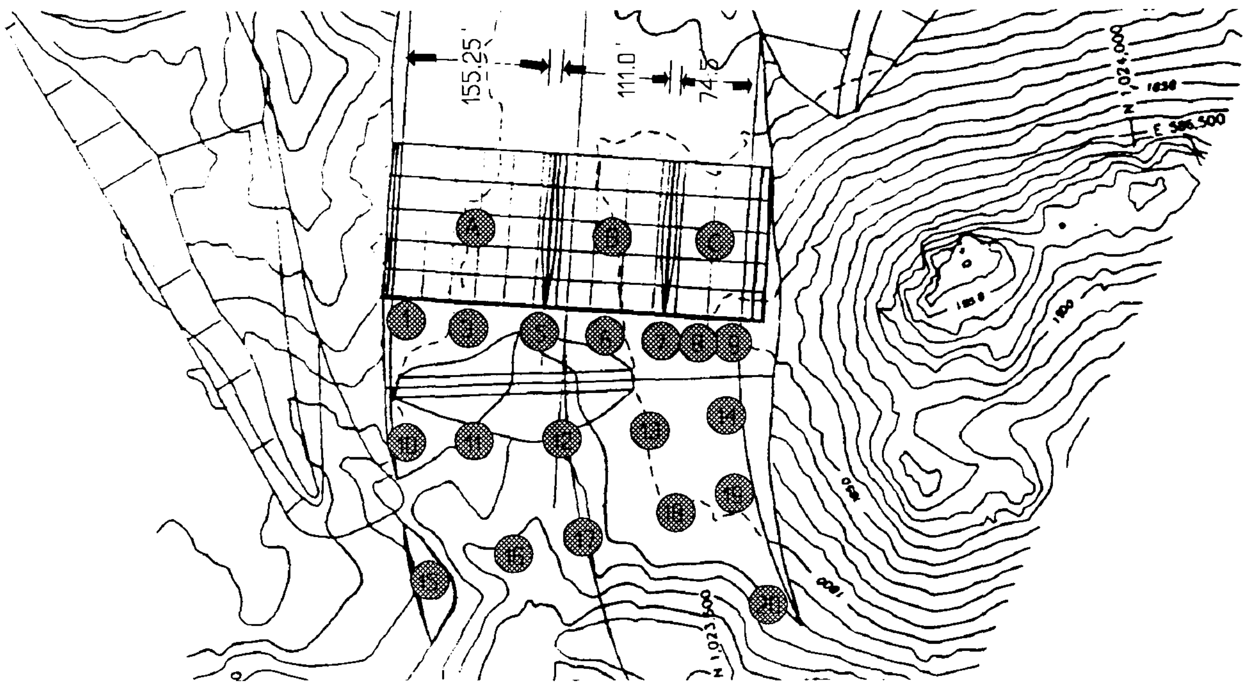


Figure 17a. — Location map for velocities measured in the fuseplug approach channel for the final design.

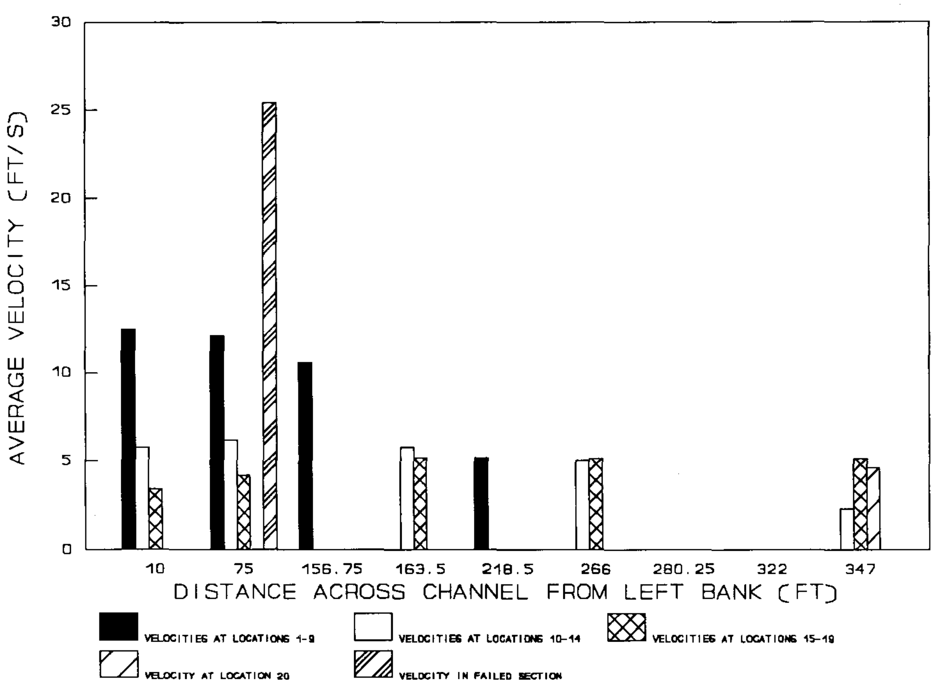


Figure 17b. — Velocity magnitudes in the approach channel and through the first failed section for the final design.

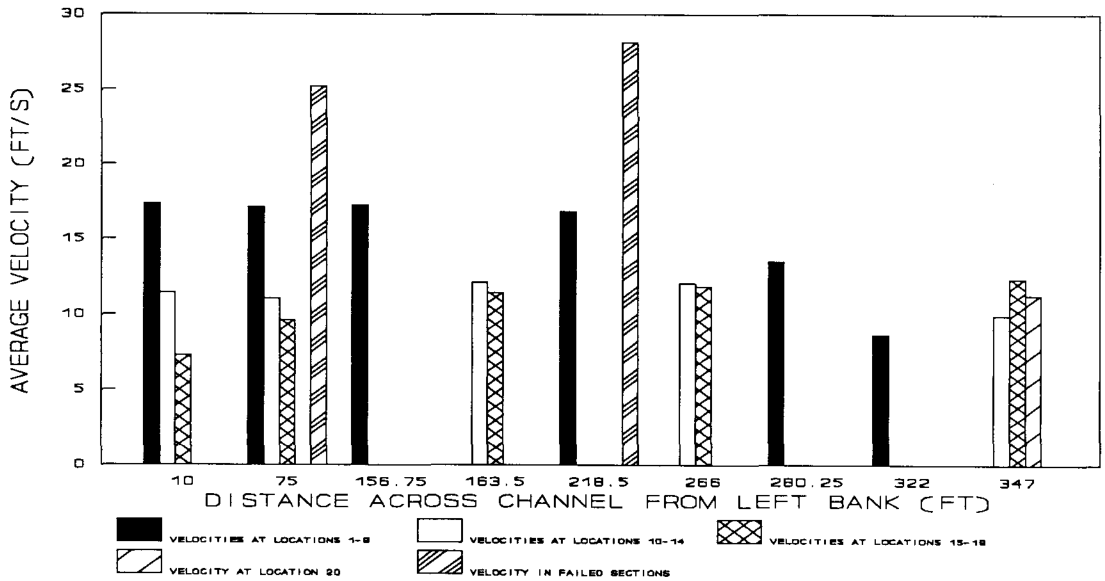


Figure 17c. — Velocity magnitudes in the approach channel and through the first two failed sections for the final design.

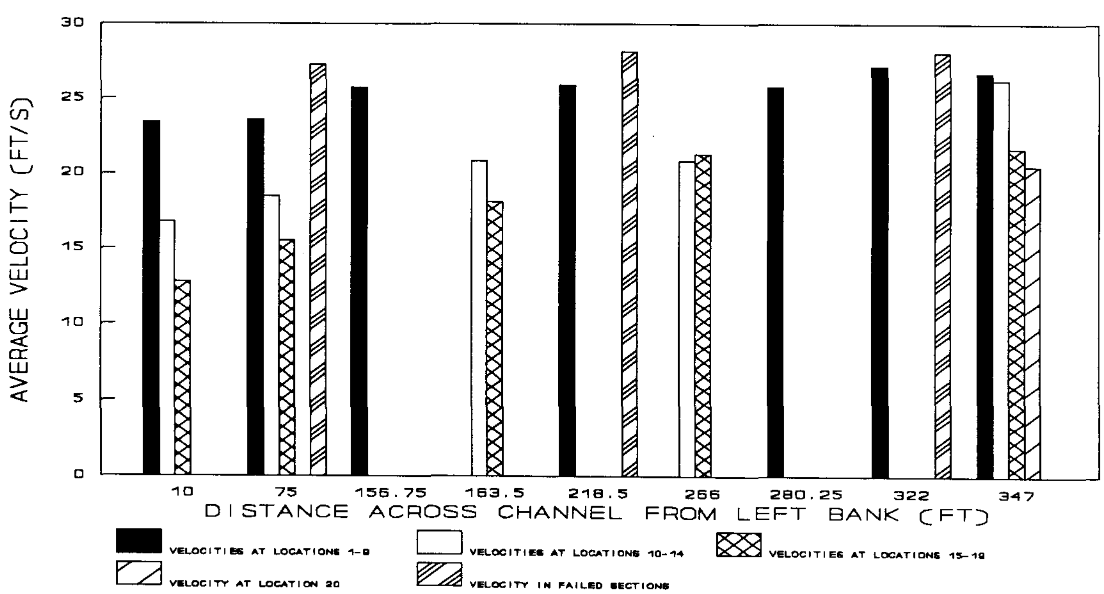


Figure 17d. — Velocity magnitudes in the approach channel and through the entirely failed section.

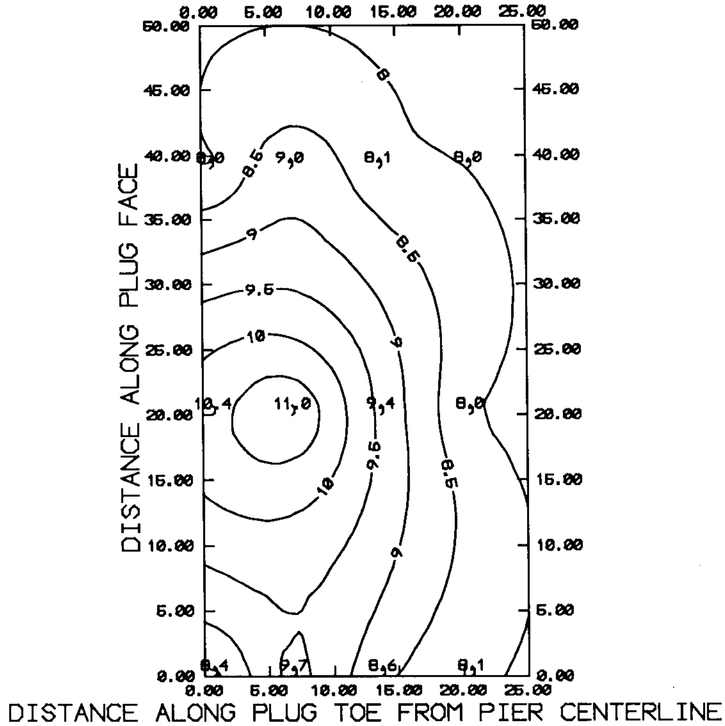


Figure 18a. — Velocity contours on the face of and normal to the second fuseplug embankment for the final design.

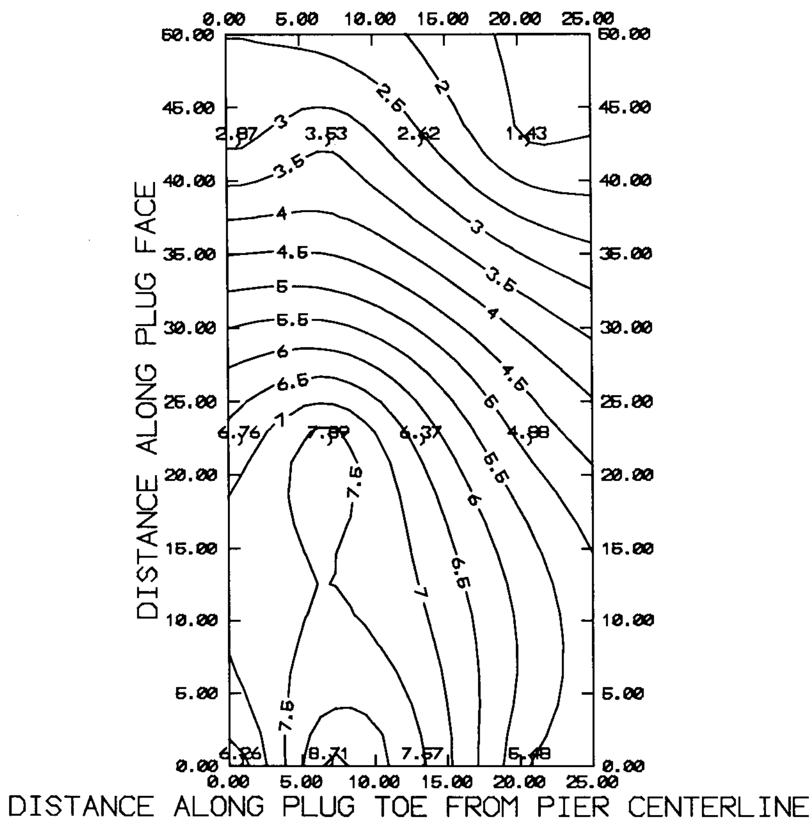


Figure 18b. — Velocity contours on the face of an normal to the third fuseplug embankment for the final design.

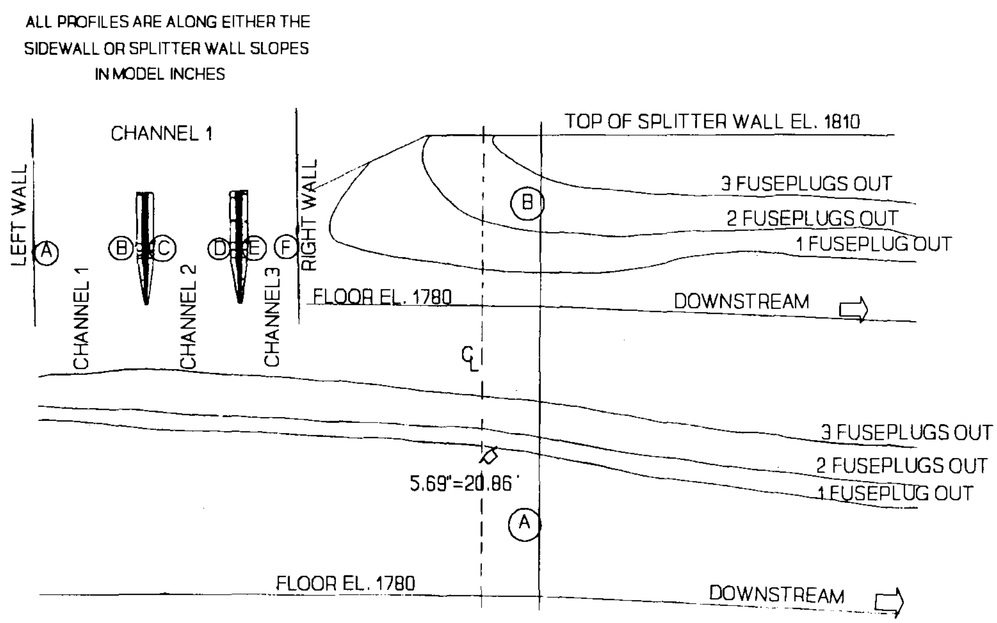


Figure 19a. — Water surface profiles measured on both sides of each fuseplug section through the control section for channel 1.

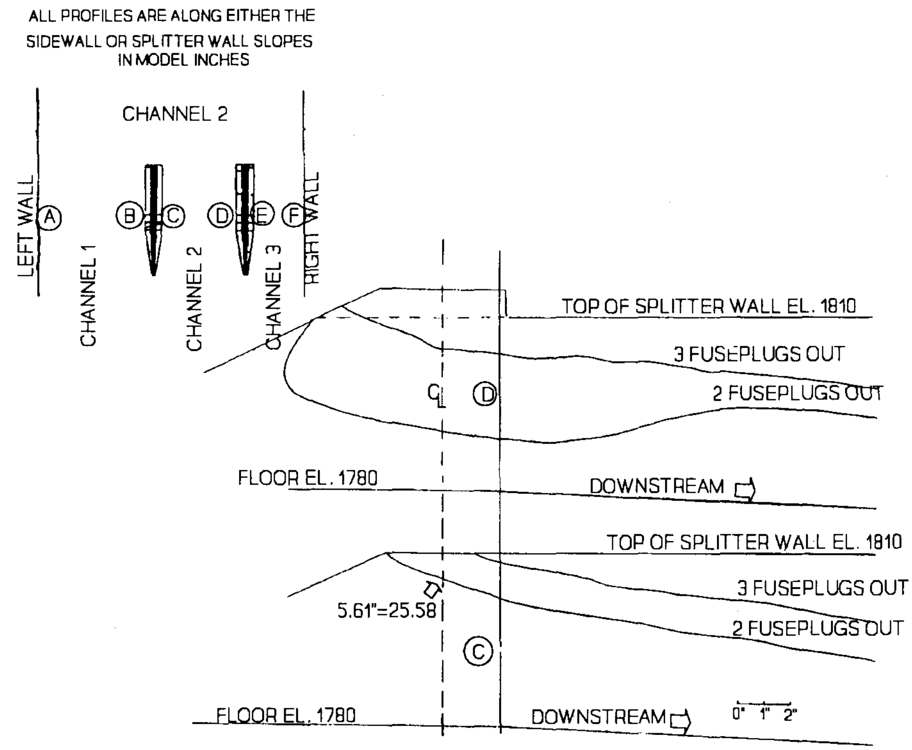


Figure 19b. — Water surface profiles measured on both sides of each fuseplug section through the control section for channel 2.

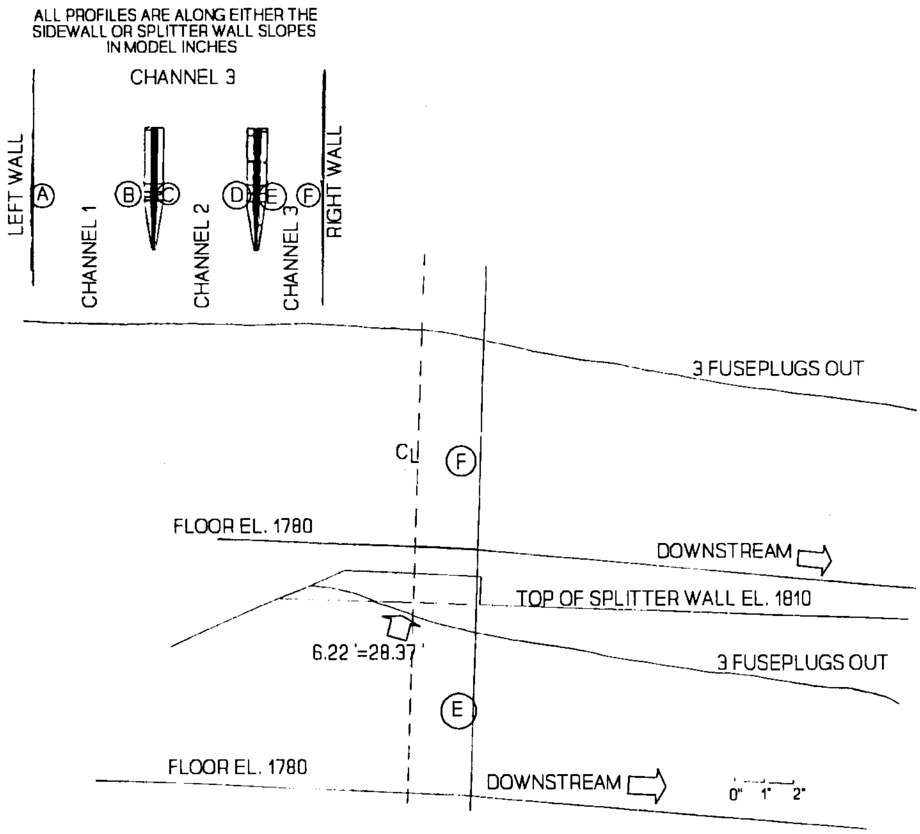
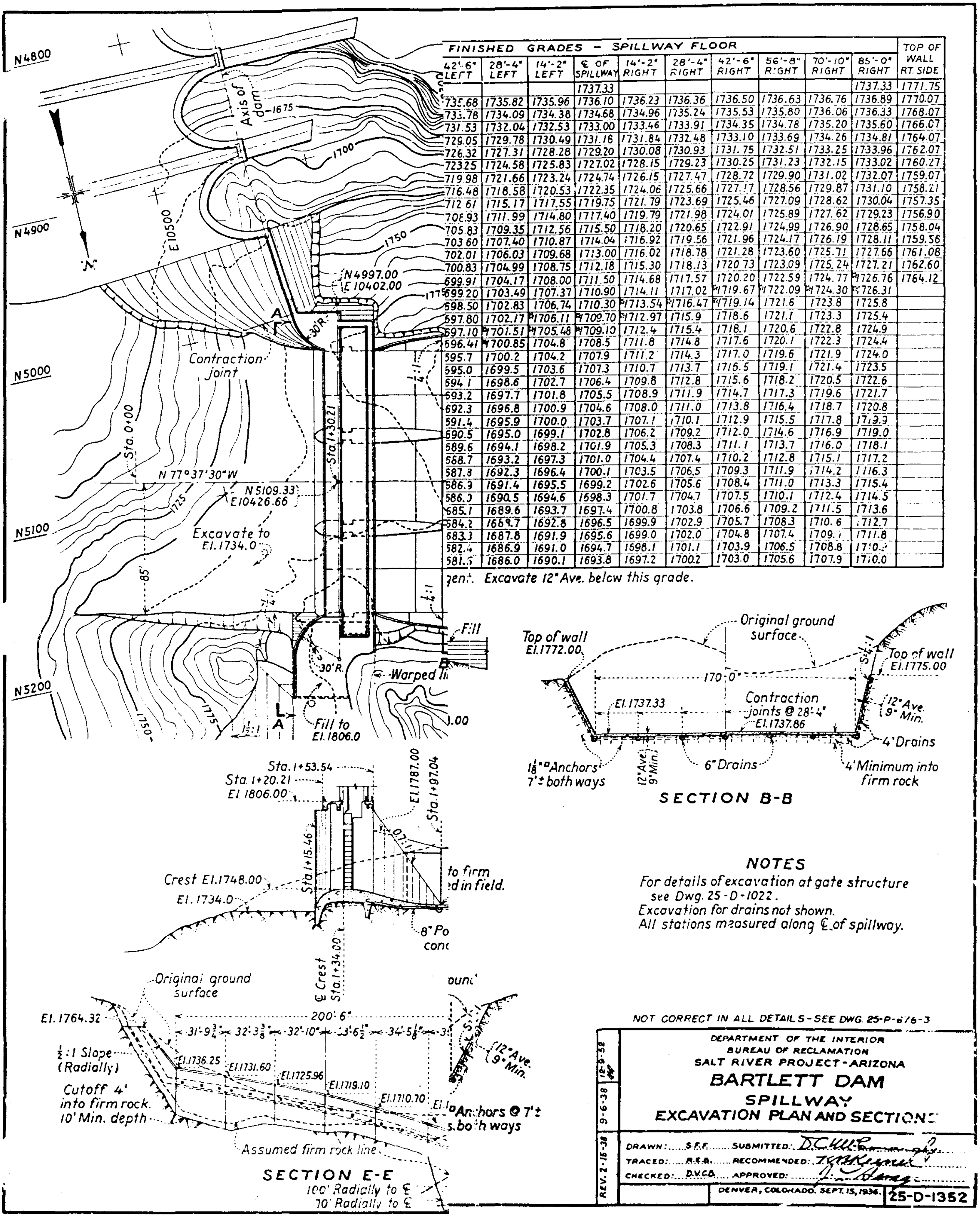


Figure 19c. — Water surface profiles measured on both sides of each fuseplug section through the control section for channel 3.



FINISHED GRADES - SPILLWAY FLOOR										TOP OF WALL
42'-6" LEFT	28'-4" LEFT	14'-2" LEFT	℄ OF SPILLWAY	14'-2" RIGHT	28'-4" RIGHT	42'-6" RIGHT	56'-8" RIGHT	70'-10" RIGHT	85'-0" RIGHT	RT. SIDE
			1737.33						1737.33	1771.75
735.68	1735.82	1735.96	1736.10	1736.23	1736.36	1736.50	1736.63	1736.76	1736.89	1770.07
733.78	1734.09	1734.38	1734.68	1734.96	1735.24	1735.53	1735.80	1736.06	1736.33	1768.07
731.53	1732.04	1732.53	1733.00	1733.46	1733.91	1734.35	1734.78	1735.20	1735.60	1766.07
729.05	1729.78	1730.49	1731.16	1731.84	1732.48	1733.10	1733.69	1734.26	1734.81	1764.07
726.32	1727.31	1728.28	1729.20	1730.08	1730.93	1731.75	1732.51	1733.25	1733.96	1762.07
723.25	1724.58	1725.83	1727.02	1728.15	1729.23	1730.25	1731.23	1732.15	1733.02	1760.27
719.98	1721.66	1723.24	1724.74	1726.15	1727.47	1728.72	1729.90	1731.02	1732.07	1759.07
716.48	1718.58	1720.53	1722.35	1724.06	1725.66	1727.17	1728.56	1729.87	1731.10	1758.21
712.61	1715.17	1717.55	1719.75	1721.79	1723.69	1725.46	1727.09	1728.62	1730.04	1757.35
708.93	1711.99	1714.80	1717.40	1719.79	1721.98	1724.01	1725.89	1727.62	1729.23	1756.90
705.83	1709.35	1712.56	1715.50	1718.20	1720.65	1722.91	1724.99	1726.90	1728.65	1758.04
703.60	1707.40	1710.87	1714.04	1716.92	1719.56	1721.96	1724.17	1726.19	1728.11	1759.56
702.01	1706.03	1709.68	1713.00	1716.02	1718.78	1721.28	1723.60	1725.71	1727.66	1761.08
700.83	1704.99	1708.75	1712.18	1715.30	1718.13	1720.73	1723.09	1725.24	1727.21	1762.60
699.91	1704.17	1708.00	1711.50	1714.68	1717.57	1720.20	1722.59	1724.77	1726.76	1764.12
699.20	1703.49	1707.37	1710.90	1714.11	1717.02	1719.67	1722.09	1724.30	1726.31	
698.50	1702.83	1706.74	1710.30	1713.54	1716.47	1719.14	1721.6	1723.8	1725.8	
697.80	1702.17	1706.11	1709.70	1712.97	1715.9	1718.6	1721.1	1723.3	1725.4	
697.10	1701.51	1705.48	1709.10	1712.4	1715.4	1718.1	1720.6	1722.8	1724.9	
696.41	1700.85	1704.8	1708.5	1711.8	1714.8	1717.6	1720.1	1722.3	1724.4	
695.7	1700.2	1704.2	1707.9	1711.2	1714.3	1717.0	1719.6	1721.9	1724.0	
695.0	1699.5	1703.6	1707.3	1710.7	1713.7	1716.5	1719.1	1721.4	1723.5	
694.1	1698.6	1702.7	1706.4	1709.8	1712.8	1715.6	1718.2	1720.5	1722.6	
693.2	1697.7	1701.8	1705.5	1708.9	1711.9	1714.7	1717.3	1719.6	1721.7	
692.3	1696.8	1700.9	1704.6	1708.0	1711.0	1713.8	1716.4	1718.7	1720.8	
591.4	1695.9	1700.0	1703.7	1707.1	1710.1	1712.9	1715.5	1717.8	1719.9	
690.5	1695.0	1699.1	1702.8	1706.2	1709.2	1712.0	1714.6	1716.9	1719.0	
689.6	1694.1	1698.2	1701.9	1705.3	1708.3	1711.1	1713.7	1716.0	1718.1	
688.7	1693.2	1697.3	1701.0	1704.4	1707.4	1710.2	1712.8	1715.1	1717.2	
687.8	1692.3	1696.4	1700.1	1703.5	1706.5	1709.3	1711.9	1714.2	1116.3	
686.9	1691.4	1695.5	1699.2	1702.6	1705.6	1708.4	1711.0	1713.3	1715.4	
686.0	1690.5	1694.6	1698.3	1701.7	1704.7	1707.5	1710.1	1712.4	1714.5	
685.1	1689.6	1693.7	1697.4	1700.8	1703.8	1706.6	1709.2	1711.5	1713.6	
684.2	1689.7	1692.8	1696.5	1699.9	1702.9	1705.7	1708.3	1710.6	1712.7	
683.3	1687.8	1691.9	1695.6	1699.0	1702.0	1704.8	1707.4	1709.1	1711.8	
682.4	1686.9	1691.0	1694.7	1698.1	1701.1	1703.9	1706.5	1708.8	1710.9	
681.5	1686.0	1690.1	1693.8	1697.2	1700.2	1703.0	1705.6	1707.9	1710.0	

Figure 20. — Bartlett Dam spillway plan and sections (25-D-1352).

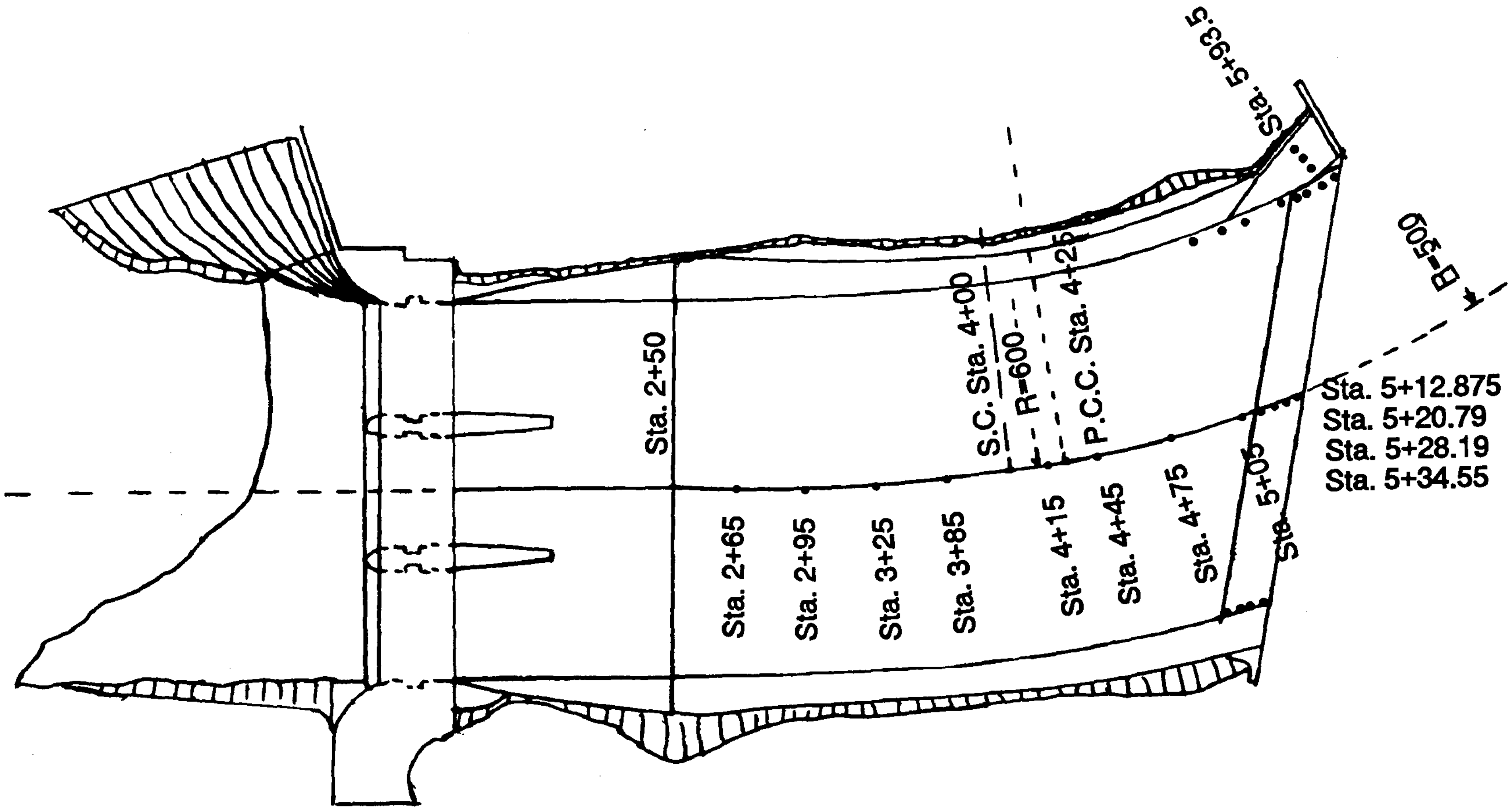


Figure 21. — Plan view of the spillway chute showing piezometer tap locations for measuring chute invert pressures.

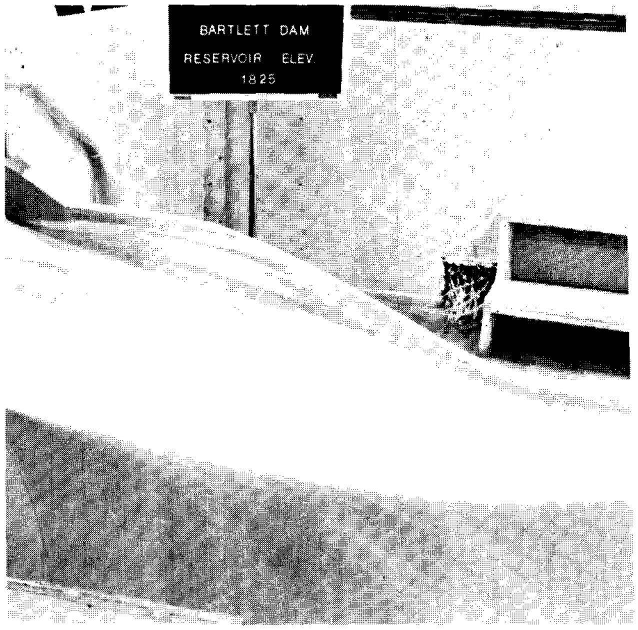


Figure 22. — Overtopping of the left and right service spillway chute walls.

Figure 22a. — Closeup view of the left wall chute overtopping.

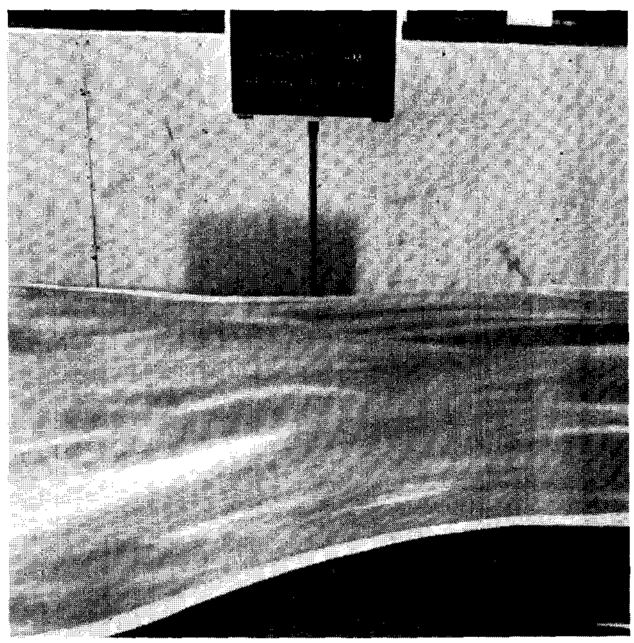


Figure 22b. — Closeup view of the right chute overtopping.

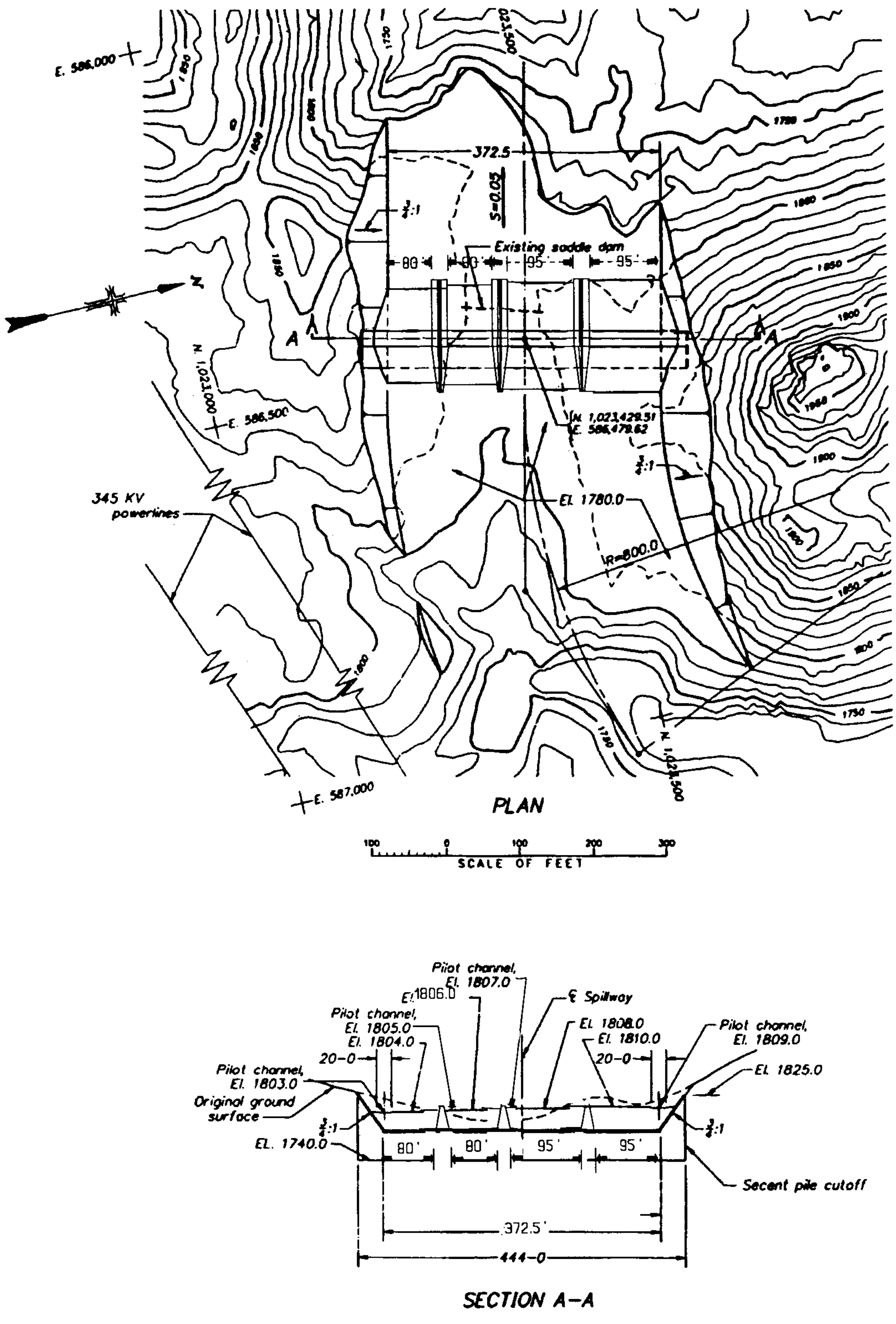


Figure 23. — Feasibility layout of the fuseplug spillway.

Figure 24. — View of the auxiliary fuseplug spillway with three sections and 3/4:1 channel side slopes.

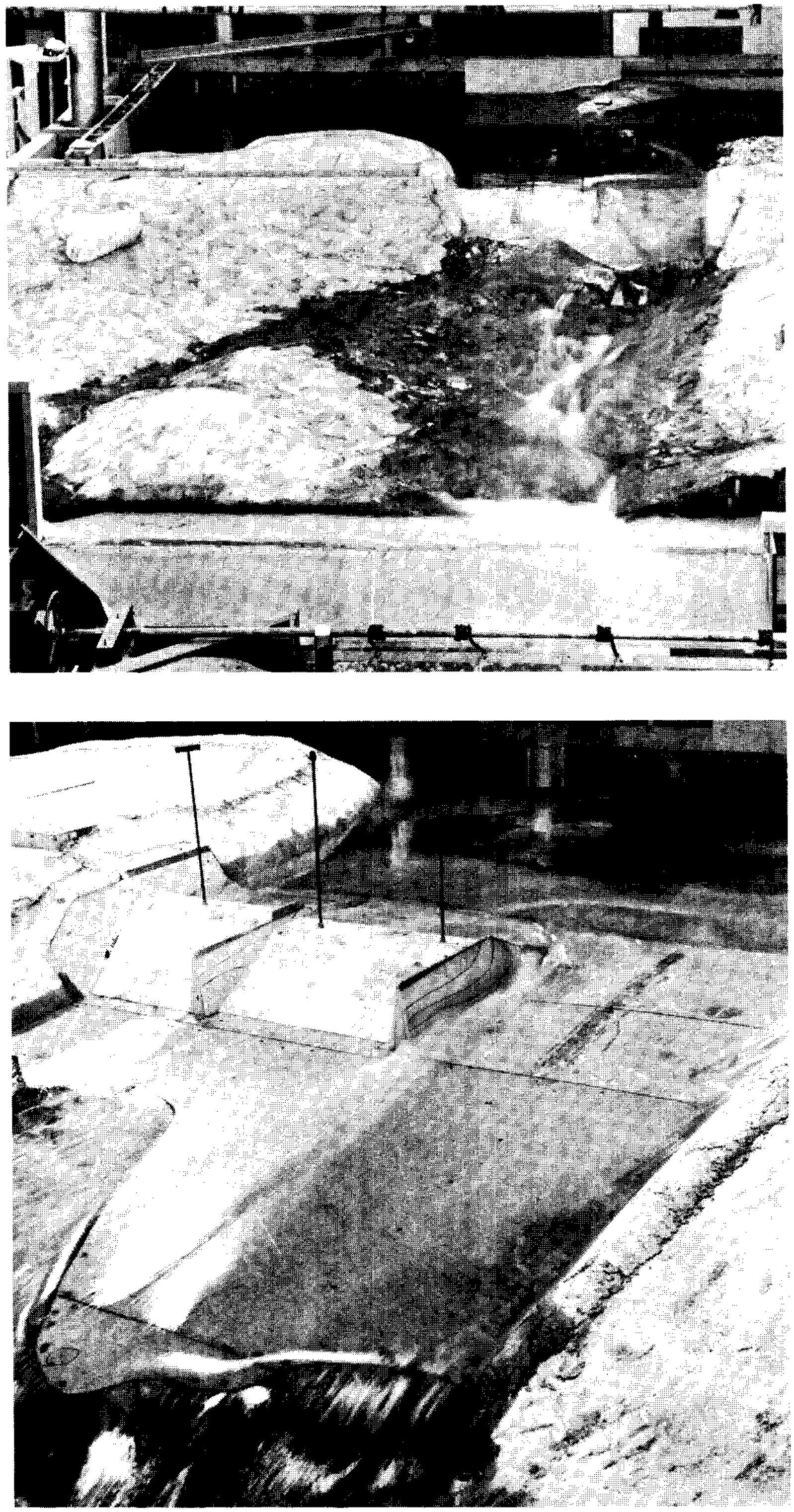


Figure 25a. — Last fuseplug section geometry tested in the model with one section failed.

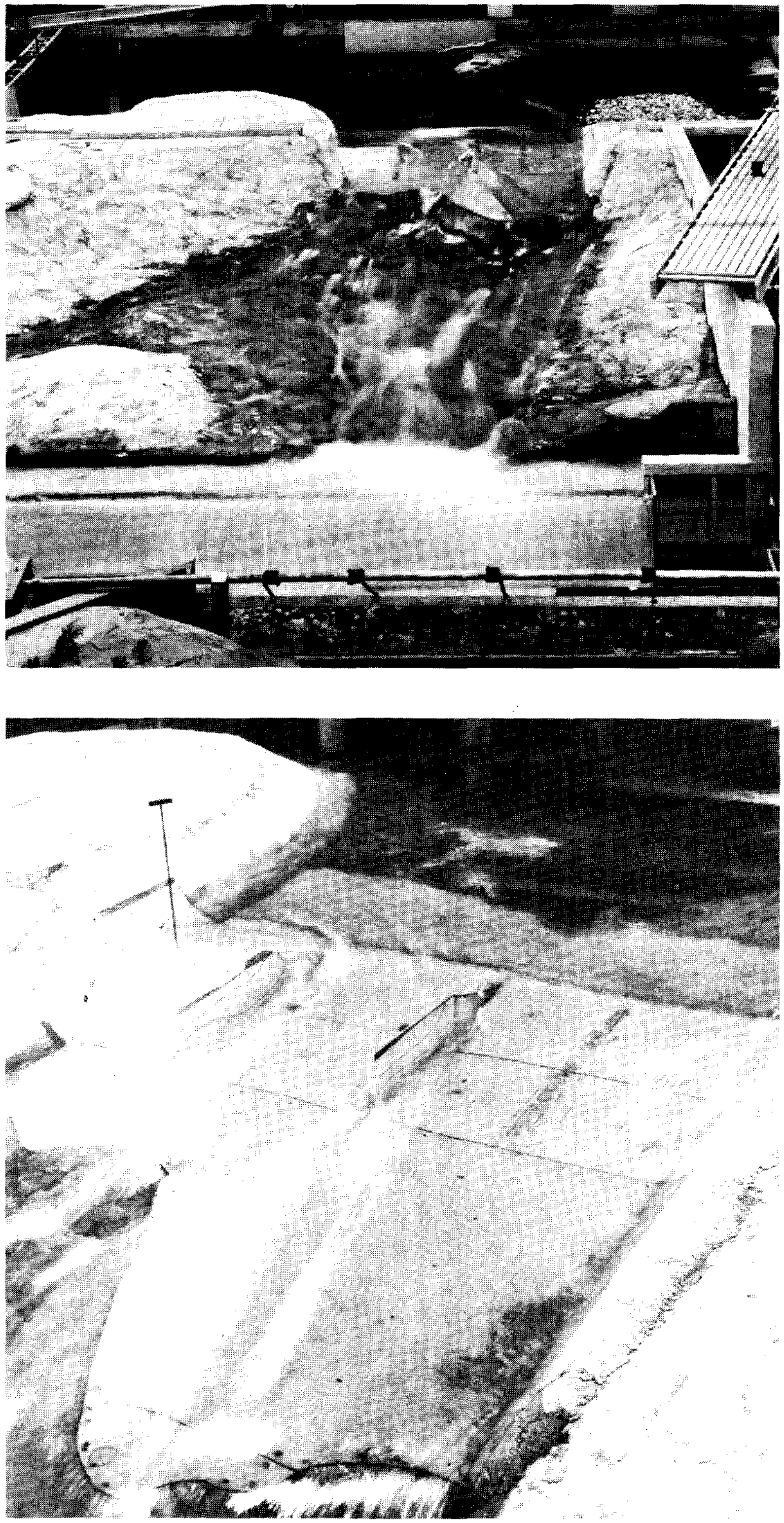


Figure 25b. — Last fuseplug section geometry tested in the model with two sections failed.

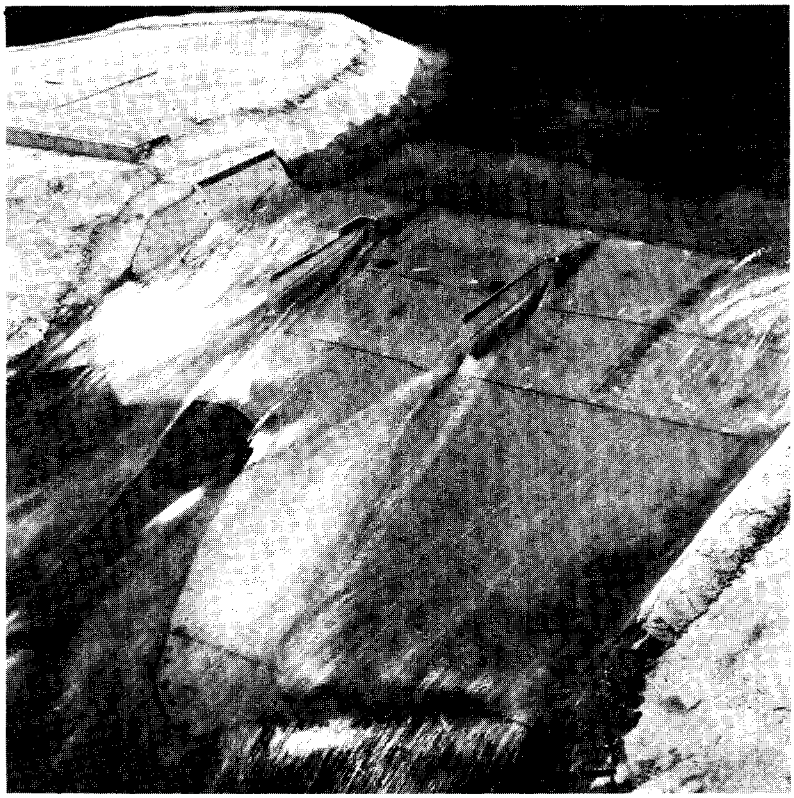


Figure 25c. — Last fuseplug section geometry tested in the model with all sections failed.

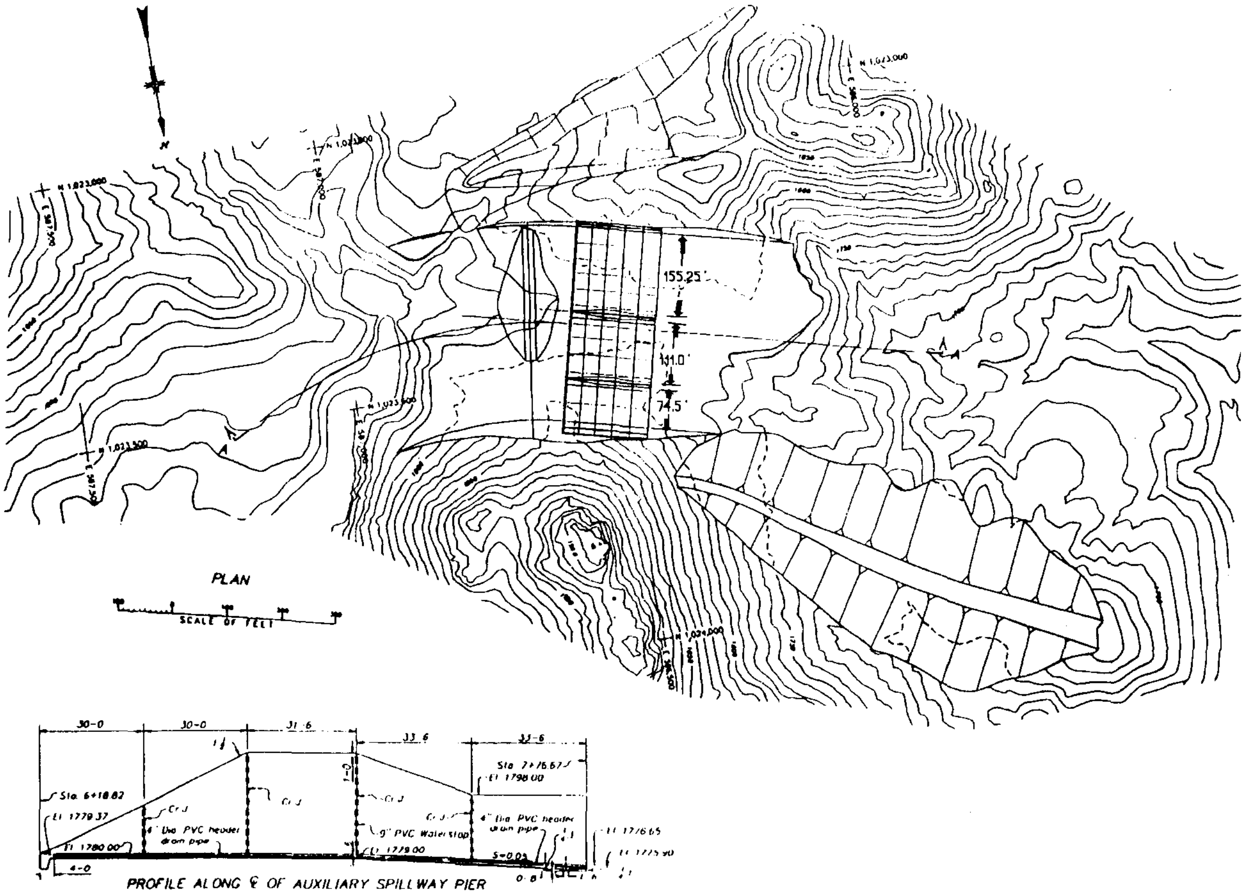


Figure 26. — Final existing fuseplug spillway design.

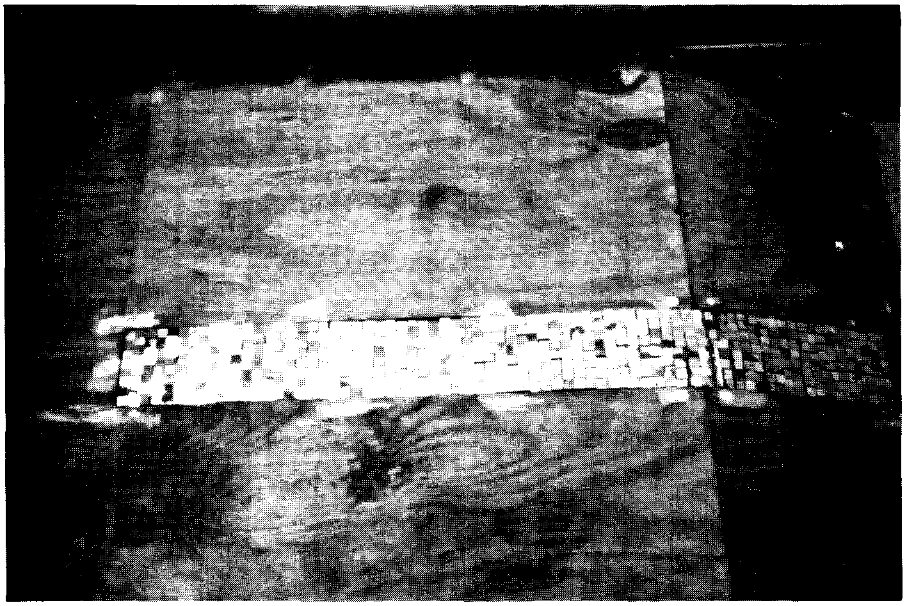


Figure 27. — Sectional model of the auxiliary spillway channel showing the aluminum blocks used to represent rock erosion (flow is from right to left).

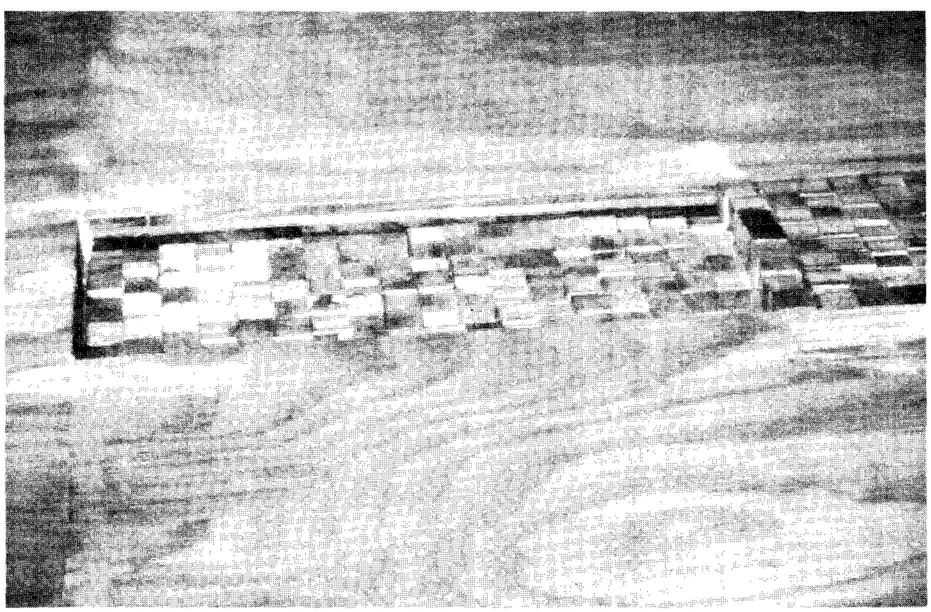


Figure 28. — Final erosion pattern after the PMF discharge across the approach channel section (flow is from right to left).

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