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Hydraulic Model Study of Pilar Dam

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HYDRAULIC MODEL STUDY OF PILAR DAM

by Leslie J. Hanna and Clifford A. Pugh

Purpose

The hydraulic model study was conducted to optimize the hydraulic performance of the proposed Pilar Dam stepped spillway and diversion works and to verify performance of proposed modifications. Hydraulic information is also provided for the design of downstream river bank stabilization.

Introduction

The site for the proposed Pilar Dam (figure 1) on the Piranga River is about 11 km southwest of Ponte Nova City in Brazil (figures 2 and 3). The dam is owned by the consortium ALCAN-FIAT and is critical to the development of Brazil's water resources. THEMAG Engineering is designing the project. The gravity concrete dam and stepped spillway will be constructed of roller-compacted concrete (RCC) with 5- by 6-meter twin culverts through the base of the dam for river diversion (figures 1 and 4). An outlet works with two 1.2-m diameter Howell-Bunger valves to the right of the spillway (figure 1) will eventually be replaced by a power plant. The dam will be approximately 67-m high with a 180-m long stepped spillway over the center.

The Water Resources Research Laboratory (WRRL), Denver, Colorado, was asked through Reclamation's Brazil project office in Brazilia to conduct the study. WRRL has considerable experience in modeling stepped spillways as a result of Reclamation's Dam Safety Program.

Conclusions

1) The performance of the proposed crest and stepped spillway designs (figure 5) was adequate up to a discharge of 2000 m^3/s and is capable of passing this flow with approximately 1 m of freeboard available between the reservoir water surface and the top of the end piers.

2) Adequate energy dissipation was provided at the toe of the dam by a 15 m wide trench excavated to elevation 465 and extending from the downstream diversion channel to the river channel. The design was confirmed for flows up to 1000 m^3/s. At flows of 2000 m^3/s, pressure fluctuations as high as 7 m were measured at the bottom of the trench. A hydraulic jump is contained in the trench for spillway flows up to 2500 m^3/s.

3) Velocities were less than 5 m/s in the downstream river channel for a spillway discharge of 1000 m^3/s. Backflow near the downstream river bank was eliminated for flows up to 1000 m^3/s as a result of converging the spillway training walls.

4) The diversion works will adequately pass a diversion flow of 235 m^3/s without overtopping the upstream cofferdam (el. 471.5 m).

5) A modified spillway design with converging side walls and an excavated energy dissipation trench at the base of the dam provides optimized hydraulic stilling performance at a discharge of 1000 m^3/s and provides adequate performance at a discharge of 2000 m^3/s.

Dr. Aluisio Pardo Canholi representing THEMAG Engineering Company (the client) visited the Water Resources Research Laboratory in July 1996 to observe the model operations with the initial design and to discuss modifications. During the visit, two modifications were made to the spillway design and a testing plan was formulated. Attachment 1 contains the meeting notes summarizing the modifications and test plan formulated during the visit.

Test Plan

The following model investigations were conducted:

1) Stepped spillway and stilling basin:

- A stage/discharge relationship was developed for the stepped spillway with discharges up to 2500 m^3/s.

- Approach conditions were studied to determine whether the design of the spillway end piers was adequate.

- Flow conditions on the stepped spillway slope were studied. This included measuring pressures down the stepped spillway for 500 m^3/s, 1000 m^3/s, 1500 m^3/s, 2000 m^3/s, and 2500 m^3/s, and measuring water surface profiles down the center of the spillway.

- Water surface profiles for determining training wall heights were measured. Required modifications to converge the side walls and divert flow away from the abutments and outlet works were also determined.

- The need for and recommended dimensions of a stilling basin at the toe of the dam were determined. This was accomplished by determining the depth

necessary to force a hydraulic jump at the toe of the dam and from observations of flow conditions in the trial basin for flows up to 2000 m^3/s.

- Velocities were measured in the downstream river channel for spillway discharges of 1000 m^3/s and 2000 m^3/s to provide hydraulic design information for bank stabilization design.

2) River diversion:

- The adequacy of the diversion channels and cofferdams to pass the design discharge of 235 m^3/s was determined.

- Flow conditions in the diversion channels under expected diversion flow rates were studied. This included velocity measurements in the upstream and downstream diversion channels and in the river channel at a discharge of 235 m^3/s.

- Stage/discharge ratings for the diversion culverts were developed for discharges up to 235 m^3/s.

- The water surface elevation required to pass a diversion discharge of 800 m^3/s was determined to estimate the dam construction progress necessary to contain a flood of this magnitude.

The Model

An undistorted geometric scale of 1:40 was used to model the entire width of Pilar Dam. The model included about 195 m[1] of the prototype topography upstream of the dam and about 330 m of downstream river topography . This scale was adequate to represent the flow over the stepped spillway and through the diversion culverts. The upstream and downstream topography included the diversion channels, the upstream cofferdam, portions of the river channel, and the dam abutments.

The Froude scaling law [Froude number = $V/(gL)^{1/2}$] was used because hydraulic performance in free surface models is primarily dependent on gravitational and inertial forces. Froude law similitude produces the following relationships between the model and the prototype:

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[1] All dimensions are given in prototype scale.

Length ratio	$L_r = 1:40$
Velocity ratio	$V_r = L_r^{1/2} = 1:6.33$
Discharge ratio	$Q_r = L_r^{5/2} = 1:10,119.29$

The spillway crest, steps, and end piers were formed from high density polyurethane with the dimensions provided by THEMAG Engineering (figure 5). The training walls were made from clear Plexiglass so that water surface profiles could be viewed through the walls, marked, and measured (figure 6).

Piezometer taps were installed 53.86 m from the inside face of the right spillway pier (measured parallel to the spillway longitudinal axis, figure 7). Taps were installed in the crest and on the first, third, and sixth steps; and then on every fourth step down the slope of the spillway to elevation 475.24 m. Two taps were installed in each step; a vertical tap was centered on the horizontal face, and a horizontal tap was centered on the vertical face (see inset on figures 13-17).

In addition, a pressure transducer was installed in the trench (see figure 31) to measure maximum fluctuating pressures at the toe of the dam. Inflows into the model were measured using the permanent laboratory Venturi meter flow measurement system. A point gage was used to measure reservoir elevations. The Swoffer velocity meter (propeller meter) was used to measure velocities in the diversion and river channels.

Results

1) Initial investigations of the original spillway design (figure 1) revealed that impacting flow on the left abutment was significant throughout the range of flows (figure 8). Considering the presence of loose material on the left abutment, the entire crest was moved 10 m to the right of the original design.

In addition, both training walls were converged at an angle of 16 degrees in the plane of the spillway to divert flow away from the abutments and outlet works. This arrangement maintains the 180-m spillway crest width to achieve the required spillway discharge capacity, then converges the walls to a width of 140 m at elevation 471.5 m (see figure 9).

2) To provide energy dissipation in the downstream river channel, a trench was installed at the toe of the dam excavated to an elevation of 465 m (figure 9). The trench extends from left diversion wall to the river channel and is 15 m wide with a 1:1 slope back to the natural topography at the downstream end of the trench. This

provides enough depth at the toe of the dam for a hydraulic jump to form for all spillway flows up to 2500 m^3/s. Figures 9 and 10 show the modified spillway and trench designs.

3) As a result of the crest relocation, the outlet works intake and tower were moved 5 m to the right of its original location so that flow over the crest will be unaffected by the intake structure.

4) Water surface profiles for flows ranging from 500- to 2500-m^3/s were determined for the spillway training walls (figure 11) and are given in tabular form in table 1. These values can be used to determine wall heights. An additional wall height up to 37 percent higher would normally be added to prevent overtopping due to bulking of the spillway flow. However, minor overtopping at very high flows may not be a concern since energy is dissipated by the wall and the dam face behind the wall will have RCC.

5) It is recommended that the right training wall be tapered into the downstream diversion culvert wall to prevent direct impact on the top surface of the culvert wall. The width of the berm on the left side of the diversion channel should also be minimized to reduce the impingement surface.

6) The discharge rating curve developed for the modified spillway (figure 12 and table 2) demonstrates that 3.04 m of head (reservoir elevation 530.04 m) is required to pass a discharge of 2000 m^3/s, leaving about 1 m of freeboard between the reservoir water surface and the top of the dam. A discharge of 2500 m^3/s does not cause overtopping of the dam and requires a reservoir elevation of 530.47 m.

7) Observations of flow conditions as well as spillway pressures measured down the slope of the spillway (figures 13 thru 17 and tables 3 and 4) demonstrate that the spillway performs adequately throughout the range of flows up to 2500 m^3/s. The pressures on the second full-sized step (step 6) indicate lower pressures (near atmospheric) on the horizontal surface. However, velocities at this location are only about 10 m/s and the cavitation index is estimated to be 1.7 which lies well above the region of incipient cavitation (indicating cavitation should not be a concern). Therefore, this design should be adequate. More gradual step transitions or setting the step edges to coincide with the ogee curve as shown in figure 18 would smooth out the pressure transition by providing a less abrupt change in slope from the crest to the spillway [6].

8) Water surface profiles down the center of the spillway match relatively closely with the pressures on the vertical pressure taps (horizontal steps) (figure 19 and table 5).

9) Approach flows around the spillway end piers are adequate for all flows up to 2000 m^3/s (figure 20) and will not require any design changes.

10) Velocities measured in the downstream diversion and river channels for spillway discharges of 1000 m^3/s and 2000 m^3/s are shown in figures 21 and 22 respectively. Table 6 shows that for a discharge of 1000 m^3/s, the maximum velocity measured in the river channel is 2.72 m/s, with a maximum of 4.3 m/s measured in the downstream diversion channel. No backflow conditions exist at a flow of 1000 m^3/s (as they did in the original spillway design). The maximum velocity for a flow of 2000 m^3/s is 5.31 m/s measured immediately downstream of the trench.

11) The discharge rating curve for diversion flows up to a maximum discharge of 235 m^3/s is shown in figure 23. Tabulated values are given in table 7. The reservoir elevation corresponding to a diversion discharge of 235 m^3/s is 470.3 m, providing 1.2 m of freeboard on the cofferdam. At a flow of 800 m^3/s, discharge through the diversion works has changed from free surface flow to orifice flow and requires a reservoir elevation of 490.3 m to the pass the flow through the diversion culverts.

12) Figure 24 shows the magnitude and direction of the velocities measured for a diversion flow of 235 m^3/s. The magnitude of each velocity measured is also listed by grid point in Table 8. Velocities measured adjacent to both the upstream (grid point 69) and downstream (grid points 26 and 27) cofferdams are quite low, ranging from .06 m/s to .32 m/s. The downstream cofferdam was simulated with rocks during these tests.

13) Pressures measured at the toe of the dam for spillway discharges of 1000 m^3/s and 2000 m^3/s are listed in Table 9. Figures 25 and 26 show the pressure traces. The average pressure was 7.1 m at 1000 m^3/s. The maximum peak-to-peak pressure fluctuations were 3.7 m for 1000 m^3/s . The standard deviation of the pressure fluctuations was 0.48 m at 1000 m^3/s, therefore two-thirds of the time the pressures are within $\pm$ 0.48 m of the average pressure. For 2000 m^3/s, the average pressure and maximum peak-to-peak pressure fluctuations were 15.5 m and 7.0 m respectively. The standard deviation at 2000 m^3/s was .93 m.

Investigations

Energy Dissipation at the Toe of the Spillway

The main objective of this analysis was to evaluate the degree of energy dissipation at the toe of the dam. The initial design did not include a stilling basin at the toe of the dam. The model was modified by adding an excavated trench to elevation 465.00 at the toe of the dam (figures 9 and 10). The trench is 15 m wide with a 1:1 slope (end sill) back to the natural topography at the downstream end of the trench. Observations of flows up to 1000 m^3/s indicate excellent performance with the trench installed (figure

27). A stable and fully contained hydraulic jump is formed at this flow. At a flow of 2000 m^3/s, performance is still good with a boil at the downstream end of the trench where the flow cascades over the natural topography (figure 28). For comparison purposes, "Design of Small Dams" was used to calculate the minimum depth and length for a type II stilling basin design, based on measured velocities entering the basin (this most closely approximates the trench and end sill configuration). For a flow of 1000 m^3/s, the minimum calculated length is 14 m to the type II basin end sill and the minimum tailwater depth required is 3.7 m. The trench provides a tailwater depth of 7 m and a length of about 20 m from the toe of the dam to the top of the trench end sill, therefore maximum energy dissipation should be achieved at this flow and was confirmed with observations of flow conditions in the model. For a flow of 2000 m^3/s, the minimum calculated length and tailwater depth for the type II basin are 27.5 m and 7 m respectively. Although the length of the trench to the end sill is short of the calculated type II length, the end sill is much higher and the tailwater depth (10.5 m) is much deeper, resulting in adequate performance. These calculations are only approximate since the trench was not designed as a type II basin, however, they demonstrate that the trench dimensions are within reason. In addition, the tailwater was lowered to elevation 468 m for both flow rates and the jump did not sweep out in either case, demonstrating that the trench will provide adequate depth even if the tailwater is lower than expected.

A pressure transducer was installed at the toe of the dam (see figure 31) at the maximum impact point to measure the fluctuating pressures for spillway discharges of 1000 m^3/s and 2000 m^3/s (figures 25 and 26). The maximum, average, and fluctuating pressures are listed in table 9. Table 9 demonstrates that the average pressures measured at the toe of the dam are nearly equal to the static pressure or tailwater depth above the surface of the trench with wave fluctuations on the static pressure. This concurs with what has been observed in past model studies and field tests.

In addition, moveable rock bedding averaging 1 m in diameter and a total depth of 2 m was installed at the toe of the dam in the trench area (figure 29). This reduced the depth of the trench by 2 meters, thereby reducing the available tailwater by 2 m and bringing the trench invert elevation to 467 m. A flow of 1000 m^3/s shifted the rock slightly away from the toe of the dam but left the remaining rock in the trench intact (figure 30). A flow of 2000 m^3/s completely washed the rock out of the trench area into the river channel and onto the downstream topography (figure 31). According to Reclamation's Monograph 25, a velocity of 5.31 m/s (maximum trench outflow velocity measured at 2000 m^3/s) is enough to move stones up to 1.14 m in diameter (figure 32) [5]. Therefore, the velocities in the trench at elevation 467 m are enough to dislodge 1-m-diameter loose rock at Q = 2000 m^3/s. The trench is actually 2 m deeper than the top of the rock during this qualitative test. If any erosion occurs in the rock at flows greater than 1000 m^3/s, the ultimate trench erosion will stabilize as the stilling basin energy dissipation reaches equilibrium. The decision to deepen the trench further or to

place a slab on the floor of the trench will depend on the geology of the rock and the extra depth of the RCC dam below the trench invert.

The stepped spillway reduces the velocity entering the basin from about 30.1 m/s (calculated velocity for a 11 $m^3/s/m$ unit discharge over a 65 m high spillway on a slope of 0.8 and a Manning's n of 0.013) to a velocity of 16.5 m/s for the 0.6-m steps (velocities measured in the model). This reduces the kinetic energy to 30 percent of the energy for a smooth concrete surface since the energy is proportional to the velocity squared (70 percent of the kinetic energy has been dissipated by the steps).

Increasing the step heights from 0.6 m to 0.9 m would decrease the kinetic energy entering the basin by an additional 10 percent [4]. This would translate to a velocity reduction from about 16.5 m/s entering the basin to about 14.85 m/s for a flow of 2000 m^3/s.

$$\text{Kinetic Energy} = V^2/2g$$

$$V_2^2/2g = .90\, V_1^2/2g;\ V_2^2 = .90V_1^2$$

$$V_2 = .95\, V_1$$

Where V_1 = Velocity entering basin (16.5 m/s, measured in the model) with a step height of 0.6 meter.

V_2 = Estimated velocity entering the basin (14.85 m/s) with a step height of 0.9 meter.

Flow over the Spillway

Early investigations revealed that the impact on the left abutment was significant throughout the range of spillway flows investigated (figure 8). To alleviate this problem while maintaining the total crest length and required discharge capacity, the spillway crest was moved 10 m to the right of the original design. In addition, both training walls were converged by 16 degrees in the plane of the spillway slope so that the flow impact would miss the left abutment as well as protect the outlet works on the right abutment. The distance along the longitudinal axis between the walls varies from 180 m where each wall meets the spillway pier and crest (el. 525.945 m) to a distance of 140 m apart at elevation 471.5 m (figure 9). This configuration allows the flow to enter the tailwater without striking the abutments and the concentrated flow at the training walls is directed into the deepest parts of the tailwater at each side of the spillway; into the diversion channel on the right side, and into the river channel on the left.

The discharge rating curve for the spillway is shown in figure 12. The figure demonstrates that at a reservoir elevation of 530.04 m the spillway will pass the required discharge of 2000 m^3/s. The discharge coefficients for the spillway crest are shown in figure 33 and were calculated as follows:

$$C = Q/ LH^{3/2}$$

where C = Discharge coefficient ($m^{1/2}/s$)
Q = Discharge (m^3/s)
H = Head above the crest (m)
L = Crest length (m)

Figure 33 shows that the discharge coefficient approaches 2.14 $m^{1/2}/s$ (3.9 $ft^{1/2}/s$) for a discharge of 2000 m^3/s. This concurs with what is demonstrated in "Design of Small Dams" for relatively short ogee shapes and large values of P/H where P is the approach flow depth [1].

Figure 11 shows water surface profiles along the converging training walls for the modified training walls at 500 m^3/s, 1000 m^3/s, 1500 m^3/s, 2000 m^3/s, and 2500 m^3/s. The water surface height was measured normal to the 0.8 to 1 slope with respect to the step edge. The step number, step elevation, and water surface height where each measurement was taken are identified in table 1. The water surface depth next to the walls is higher than over the center of the spillway because the flow rolls as a result of the converging walls. The maximum water surface height occurs at a flow of 2500 m^3/s near the bottom of the spillway (steps 62 and 66) and is about 4.57 m measured normal to the slope.

Because a Froude scale model does not accurately simulate the affects of aeration in the flow depth over the spillway, additional elevation is normally added to the height of the training walls if overtopping is not allowed at large flows. Reclamation's Engineering Monograph 41 (EM41) was used to determine the bulked or aerated depth in the spillway by relating it to the nonaerated depth that is measured in the model by [2]:

$$d_b/d = 1/(1-C)$$

where d_b = bulked flow depth
d = flow depth measured
C = mean air concentration

The mean air concentration (27 percent) was determined from EM41 which relates the air concentration to the distance down the slope. This analysis shows that d_b/d is 1.37, therefore an additional 37 percent in depth would be added to wall heights to prevent

overtopping of the training walls. However, since the flow in the model is partially aerated, a value less than 37 percent would be adequate to prevent overtopping. In addition, minor overtopping at large flows may be acceptable since the dam face behind the wall is RCC.

It is recommended that the right training wall be tapered into the downstream diversion culvert wall in order to prevent direct impingement on the top surface of the culvert wall. The width of the berm on the left side of the diversion channel near the toe of the dam should also be minimized to reduce the impingement surface.

Piezometer taps were installed in the spillway steps to measure pressures on the spillway (figure 7). Figures 13 through 17 show spillway pressures measured for each flow rate with the vertical and horizontal piezometer taps located on the spillway steps. Tables 3 and 4 identify the step number, step elevation, and corresponding pressure for each flow rate. Figures 13 through 17 show that as the discharge is increased above 1000 m^3/s, low pressures pull more flow over the crest. This causes the jet hitting the first large step (step 5) to flow horizontally above the next step (step 6) thereby causing low pressure on the horizontal face (or vertical tap) and higher pressure at the horizontal tap where the flow reattaches at the step (figure 34). This phenomena causes some air to be trapped in the inset area between steps 5 and 6 in the model. However, the cavitation index at this location is estimated to be 1.7 which lies well above the region of incipient cavitation [3]. Therefore, cavitation should not be a concern. Maximum velocity in this area would only be about 10 m/s. In order to eliminate the low pressure zone between steps 5 and 6, a more gradual transition is needed between the crest and the spillway slope. This could be achieved by designing the crest so that the tips touch the projection of the natural nappe trajectory as shown in figure 18. This shape was confirmed in model studies of Monksville Dam [6].

Figures 13 through 17 show that aeration begins to occur at about step 22 where the horizontal taps read near atmospheric pressure and then fluctuate around atmospheric down the remainder of the slope. The fluctuating pressures down the slope of the spillway are consistent with observations of the fluctuating water surface. Figure 19 shows the water surface profiles over the middle of the spillway and the vertical pressures for flows of 1000 m^3/s and 2000 m^3/s. The two curves are relatively close within the accuracy of measuring the fluctuating pressures on the stepped spillway surface.

Approach flows around the piers are adequate for all flows up to 2000 m^3/s and should not require any design changes. The depth of drawdown around the pier was measured normal to the step edge at the first step as flow comes around the pier and begins dropping over the spillway. The values are listed in table 10. The depth of drawdown is defined as the difference between the water surface elevation measured at the first step near the middle of the spillway and the water surface elevation at the

first step next to the pier. No separation occurs next to the spillway piers for any of the flows tested.

Flow Velocities Downstream of the Spillway

A grid was set up downstream of the spillway to measure velocities in the downstream river and diversion channels. Figures 21 and 22 show the magnitude and direction of velocities measured at each grid point for discharges of 1000 m^3/s and 2000 m^3/s respectively. The magnitude of each velocity is indicated by the length of the arrow relative to the scale shown. The magnitude of the velocity for each grid point is also listed in table 6.

Flow conditions were optimized for a spillway discharge of 1000 m^3/s as indicated by the flow field shown in figure 21. The spillway modifications eliminated the backflow conditions along the left bank which had been observed for the initial design at this discharge. At a flow of 2000 m^3/s, some backflow occurs near the left river bank immediately downstream of the spillway. This is the result of flow passing over the island downstream from the trench. Any problems associated with this flow condition should be relatively minor and would only occur for very large floods.

Velocities in the downstream diversion channel (as indicated by grid point number 38) remain relatively low (less than 1 m/s) even at a spillway discharge of 2000 m^3/s. This is because the majority of the flow is diverted into the river channel due to the berm that separates the spillway trench from the diversion channel.

Diversion Flows

The discharge rating curve for diversion flows up to a maximum discharge of 235 m^3/s is shown in figure 23. Tabulated values are given in table 7. The reservoir elevation corresponding to a diversion discharge of 235 m^3/s is 470.3, allowing 1.2 m of freeboard on the cofferdam 9 (figure 35). In addition, a reservoir elevation of 490.3 m is required to pass a diversion flow of 800 m^3/s.

Velocities were measured in the upstream and downstream diversion channels, in the river channel, and next to the cofferdams for a diversion discharge of 235 m^3/s; at locations where velocities were sufficient to be measured. Figure 24 shows the magnitude and direction of the velocities measured. The magnitude of the velocity is indicated by the length of the arrow relative to the scale shown. The magnitude of each velocity measured is also listed by grid point in table 8. Velocities measured adjacent to both the upstream (grid point 69) and downstream (grid points 26 and 27)

cofferdams are quite low, ranging from .06 m/s to .32 m/s . The downstream cofferdam was simulated during these tests by piling rocks at the proper location.

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Table 1. Training Wall Water Surface Profiles

STEP #	Elevation (m)	Water Surface Height Measured Normal to the .8 to 1 Slope (m)				
		Q = 500 m³/s	Q = 1000 m³/s	Q = 1500 m³/s	Q = 2000 m³/s	Q = 2500 m³/s
1	525.66	0.64	1.27	1.78	2.03	2.29
2	525.31	0.76	1.52	2.03	2.29	2.54
3	524.96	0.76	1.52	2.03	2.41	2.79
4	524.56	0.89	1.65	2.03	2.41	2.79
5	524.11	1.02	1.65	2.29	2.54	2.92
6	523.51	1.14	1.78	2.41	2.67	3.05
10	521.11	1.14	1.84	2.41	2.67	3.05
18	516.31	1.27	1.78	2.16	2.29	2.54
22	513.91	1.65	2.10	2.29	2.54	2.41
26	511.51	1.78	2.35	2.54	2.79	2.92
30	509.11	2.03	2.54	2.92	3.05	3.30
34	506.71	2.16	2.67	3.05	3.30	3.56
38	504.31	2.16	1.46	3.18	3.49	3.68
42	501.91	2.41	3.05	3.30	3.68	3.94
46	499.51	2.41	3.18	3.56	3.81	4.19
50	497.11	2.41	3.30	3.68	3.94	4.32
54	494.71	2.54	3.30	3.81	3.94	4.32
58	492.31	2.54	3.43	4.07	4.07	4.45
62	489.91	2.54	3.43	4.07	4.07	4.57
66	487.51	2.54	3.56	4.07	4.07	4.57

Table 2. Spillway discharge as a function of head over the crest using curve fit results.

Head (m)	Discharge (m^3/s)
0.00	0.00
0.10	6.21
0.20	20.06
0.30	39.82
0.40	64.76
0.50	94.45
0.60	128.56
0.70	166.84
0.80	209.11
0.90	255.19
1.00	304.96
1.10	358.29
1.20	415.09
1.30	475.25
1.40	538.70
1.50	605.36
1.60	675.16
1.70	748.05
1.80	823.96
1.90	902.85
2.00	984.66
2.10	1069.34
2.20	1156.86
2.30	1247.17
2.40	1340.23
2.50	1436.02
2.60	1534.49
2.70	1635.61
2.80	1739.35
2.90	1845.69
3.00	1954.59
3.10	2066.02
3.20	2179.97
3.30	2296.41
3.40	2415.31
3.50	2536.66

Table 3. Pressure Readings for Spillway Vertical Piezometer Taps

Step Number	Elevation (m)	Pressure (m)				
		Q = 500 m³/s	Q = 1000 m³/s	Q =1500 m³/s	Q = 2000 m³/s	Q = 2500 m³/s
A	526.76	1.34	1.09	0.48	0.36	-0.00
*B	527.00	0.85	0.79	0.49	0.67	0.12
1	525.66	0.97	1.10	1.34	1.34	1.34
3	524.96	0.94	1.19	1.43	1.31	1.43
6	523.51	0.69	0.44	0.44	0.32	0.44
10	521.11	0.89	1.38	1.74	1.62	1.62
14	518.71	0.61	1.22	0.97	1.09	1.46
18	516.31	1.67	1.79	1.91	2.03	2.28
22	513.91	1.99	1.99	2.48	2.36	2.60
26	511.51	1.83	2.20	2.44	2.56	2.69
30	509.11	2.16	2.40	2.89	2.65	2.89
34	506.71	1.63	2.73	3.34	3.58	3.70
38	504.31	0.98	1.35	1.96	1.84	1.84
42	501.91	1.07	1.55	2.16	2.04	2.41
50	497.11	1.60	2.33	2.82	3.30	3.91
54	494.71	1.44	1.80	2.17	2.17	2.53
58	492.31	1.15	1.40	1.76	1.89	2.13
62	489.91	0.63	0.63	0.87	0.87	0.87
66	487.51	1.44	1.93	2.42	2.29	2.66
70	485.11	0.91	1.16	1.40	1.52	1.65
74	482.71	1.00	1.36	1.73	2.46	2.95
78	480.31	0.84	1.08	1.08	1.20	1.45
82	477.91	0.68	0.80	1.04	1.16	1.41
86	475.51	0.88	1.00	1.12	1.25	1.49

* Located at Crest Centerline

Table 4. Pressure Readings for Spillway Horizontal Piezometer Taps

Step Number	Elevation (m)	Pressure (m)				
		Q = 500 m³/s	Q = 1000 m³/s	Q =1500 m³/s	Q = 2000 m³/s	Q = 2500 m³/s
1	525.49	0.66	0.91	1.03	1.03	1.15
3	524.76	0.29	0.29	0.41	0.41	0.41
6	523.21	0.86	0.99	1.23	1.23	1.35
10	520.81	0.46	0.82	0.95	0.95	1.19
14	518.41	0.54	0.66	0.79	0.79	1.03
18	516.01	0.26	0.50	0.62	0.75	0.75
22	513.61	0.34	-0.02	-0.02	0.22	0.22
26	511.21	0.30	0.18	0.06	-0.19	-0.06
30	508.81	0.14	0.02	0.14	-0.35	0.02
34	506.41	-0.02	-0.14	-0.14	-0.14	-0.14
38	504.01	0.43	0.55	0.67	0.67	0.79
42	501.61	0.63	0.76	0.88	0.88	1.00
46	499.22	0.46	0.46	0.46	0.46	0.46
50	496.81	0.19	0.19	0.19	0.19	0.19
54	494.41	0.52	0.64	0.64	0.64	0.64
58	492.01	0.36	0.36	0.36	0.23	0.36
62	489.61	0.56	0.68	0.68	0.68	0.68
66	487.21	0.28	0.28	0.40	0.28	0.28
70	484.81	0.48	0.60	0.73	0.73	0.73
74	482.41	-0.17	-0.29	-0.29	0.20	0.20
78	480.01	-0.57	-0.45	-0.33	0.53	0.65
82	477.61	-0.37	-0.37	-0.37	-0.12	-0.12
86	475.24	0.42	0.54	0.66	0.66	0.91

Table 5. Vertical water surface component compared with vertical pressure measured above step surface.

Step Number	Discharge = 1000 m³/s		Discharge = 2000 m³/s	
	Vertical Piezometer Pressure (m)	*Vertical Water Surface Component (m)	Vertical Piezometer Pressure (m)	*Vertical Water Surface Component (m)
3	1.19	1.20	1.31	1.90
6	0.44	1.50	0.32	1.70
10	1.38	1.40	1.62	1.60
14	1.22	1.30	1.09	1.20
18	1.79	1.30	2.03	1.20
22	1.99	1.30	2.36	1.20
26	2.20	1.30	2.56	1.40
30	2.40	1.50	2.65	1.40
34	2.73	1.30	3.58	1.60
38	1.35	1.30	1.84	1.80
42	1.55	1.60	2.04	1.80
50	2.33	1.20	3.30	1.70
54	1.80	1.30	2.17	1.70
58	1.40	1.40	1.89	1.85
62	0.63	1.00	0.87	1.30
66	1.93	1.10	2.29	1.30
70	1.16	1.20	1.52	1.50
74	1.36	1.35	2.46	1.80

* The values listed were interpolated from the measured values (of the fluctuating water surface) to coincide with the location of the vertical piezometer measurements.

Table 6. Velocities measured downstream of spillway during spillway flows.

Q = 1000 m³/s		Q = 2000 m³/s	
Grid Position	Velocity (m/s)	Grid Position	Velocity (m/s)
1	0.44	1	0.82
2	1.39	2	2.85
3	1.20	3	2.78
4	1.45	4	4.62
5	1.33	5	4.81
6	1.08	6	1.83
7	1.20	7	3.67
8	0.95	8	3.04
9	1.26	9	0.82
10	0.38	10	0.76
11	1.01	11	0.70
12	0.44	12	0.38
13	1.26	13	0.63
14	1.26	14	0.13
15	1.08	15	0.57
16	1.71	16	1.20
17	0.76	17	0.06
18	0.89	18	0.19
19	1.08	19	0.38
20	1.52	20	1.01
21	0.89	21	0.25
22	0.00	22	0.00
23	0.70	23	0.06
24	1.20	24	1.14
25	1.14	25	0.25
26	0.82	26	0.51
27	0.70	27	0.25
28	0.00	28	0.95
29	2.72	29	0.25
30	1.39	30	0.57
31	1.33	31	1.08
32	1.39	32	0.57
33	2.09	33	2.28
34	1.33	34	0.89
35	2.66	35	1.33
36	2.40	36	1.26
37	0.32	37	4.93
38	0.38	38	0.82
39	0.95	39	2.15
40	1.20	40	1.58
41	1.58	41	1.58
42	1.45	42	1.26
43	0.82	43	0.19
44	0.70	44	1.71
45	2.21	45	0.89
46	0.82	55	1.39
47	1.90	56	2.85
48	3.35	57	1.58
49	1.01	58	1.45
50	4.30	59	3.35
51	2.53	60	2.72
52	1.14	61	3.79
53	0.32	62	5.31
54	1.33	63	4.05
		64	5.25
		65	5.00
		66	4.17
		67	0.57

Table 7. Diversion discharge as a function of reservoir elevation using curve fit results.

Discharge (m^3/s)	Reservoir Elevation (m)
0.00	464.00
0.54	464.20
1.82	464.40
3.72	464.60
6.17	464.80
9.14	465.00
12.60	465.20
16.52	465.40
20.89	465.60
25.71	465.80
30.94	466.00
36.59	466.20
42.64	466.40
49.08	466.60
55.92	466.80
63.13	467.00
70.72	467.20
78.68	467.40
87.00	467.60
95.68	467.80
104.72	468.00
114.10	468.20
123.83	468.40
133.90	468.60
144.31	468.80
155.05	469.00
166.13	469.20
177.53	469.40
189.26	469.60
201.31	469.80
213.68	470.00
226.37	470.20
239.37	470.40

Table 8. Velocities measured during diversion flow rate 235 m^3/s .

Grid Point	Velocity (m/s)
8	.76
10	1.77
11	1.33
12	.82
13	.32
14	.19
15	.44
16	1.2
26	.19
27	.06
37	2.72
38	3.42
68	1.14
69	.32
70	1.45
71	3.48

Table 9. Pressures measured at the toe of the spillway.

Discharge (m^3/s)	Average Pressure (m)	Maximum Pressure (m)	Maximum Peak-to-Peak Pressure (m)	Standard Deviation (m)	Tailwater Depth (m)
1000	7.1	9.4	3.7	0.48	6.5
2000	11.2	15.5	7.0	0.93	10.5

Table 10. Drawdown depth measured normal to the first spillway step.

Discharge (m^3/s)	Drawdown Depth (m)
500	0
1000	.13
1500	.38
2000	.25
2500	.51

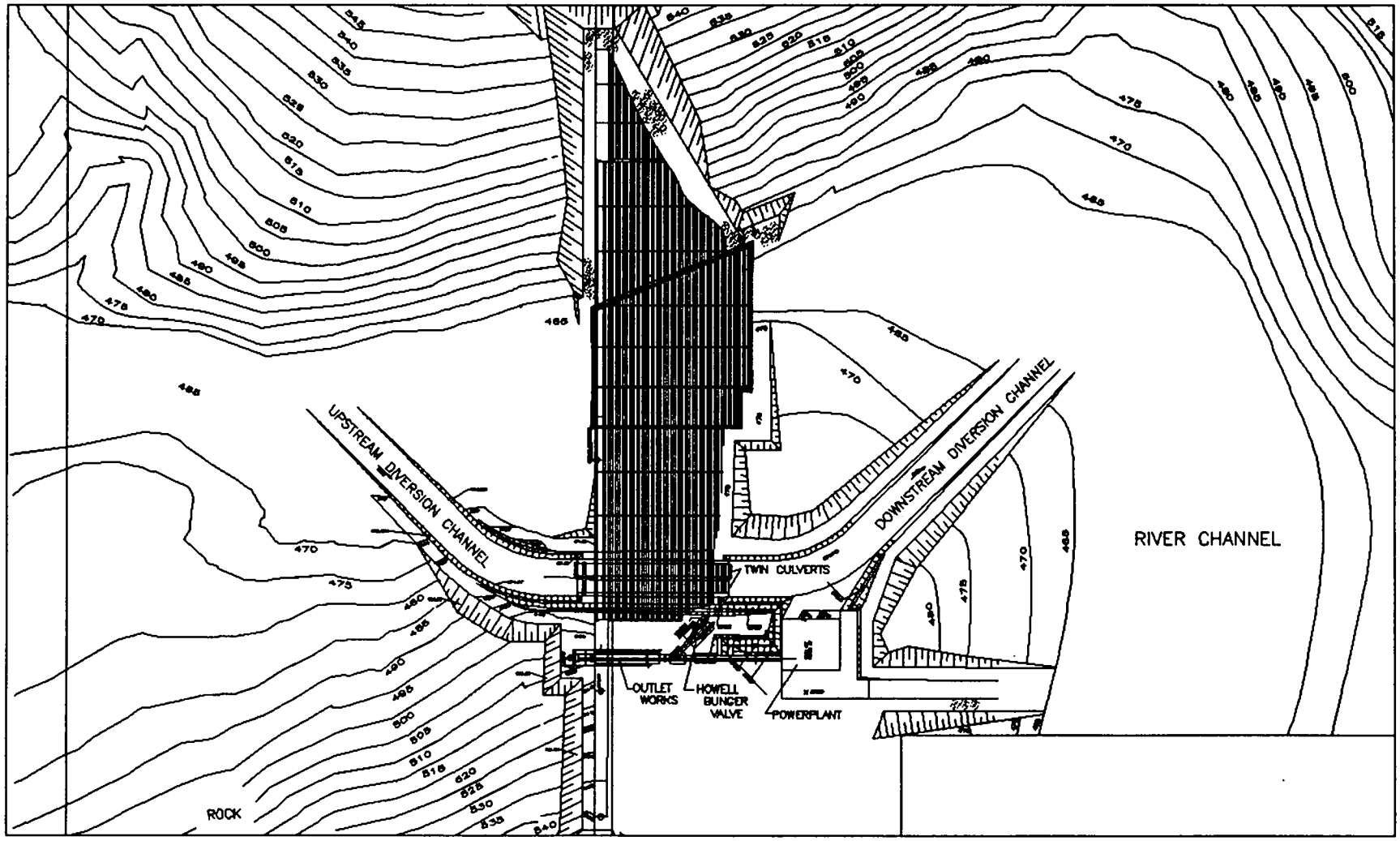


Figure 1. Outline of the model extents used for the Pilar Dam model study.

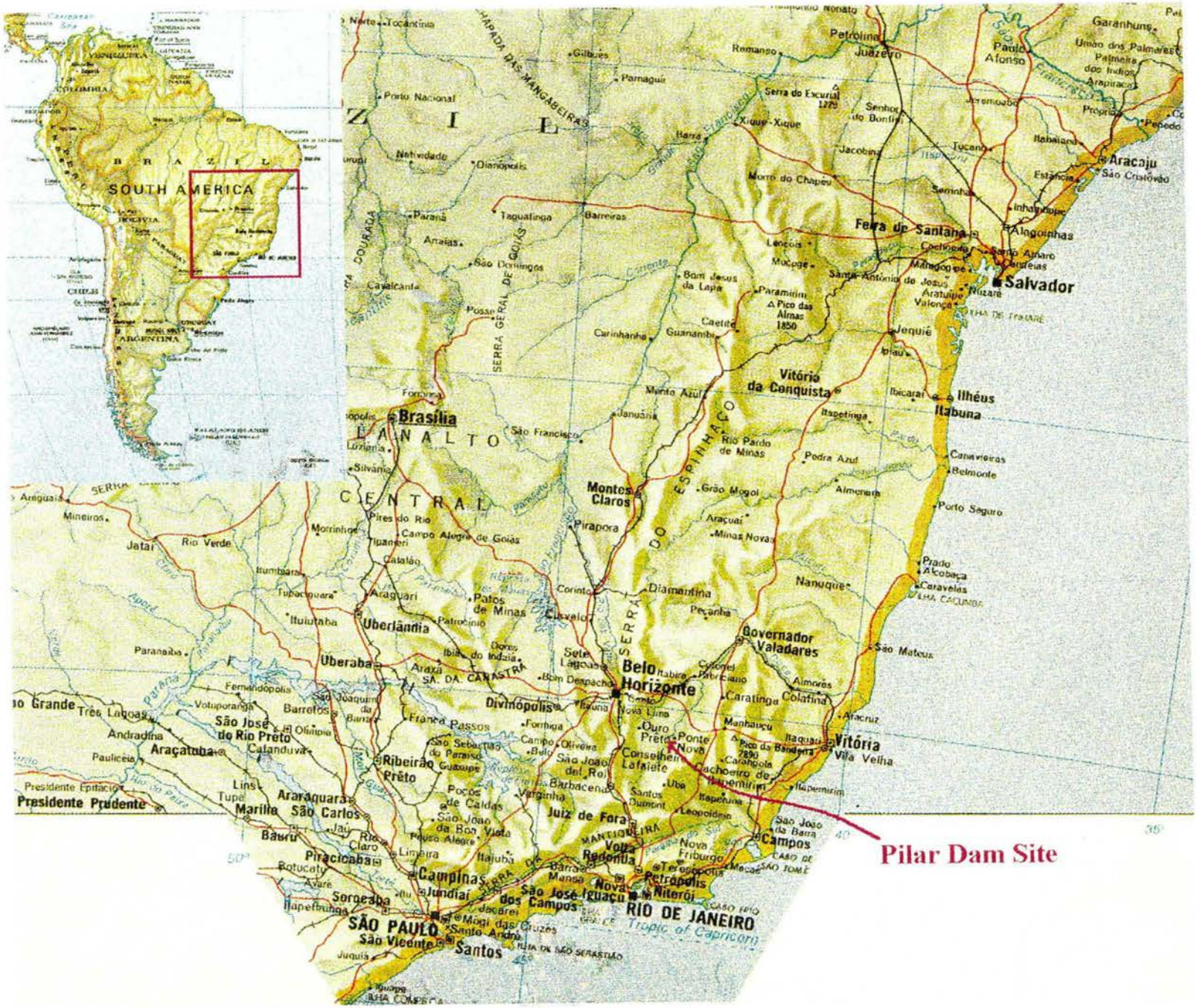


Figure 2. Pilar Dam site location.

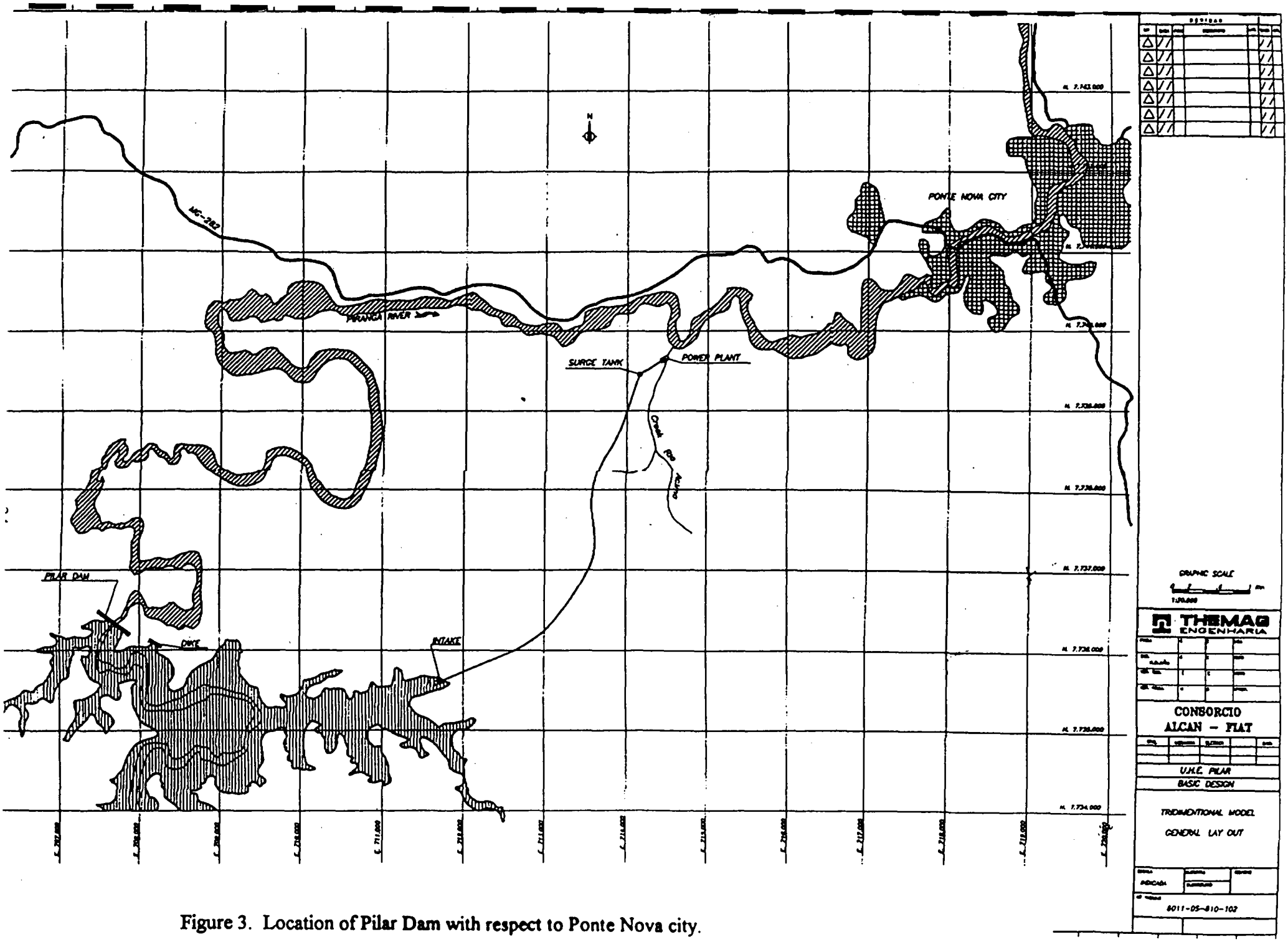
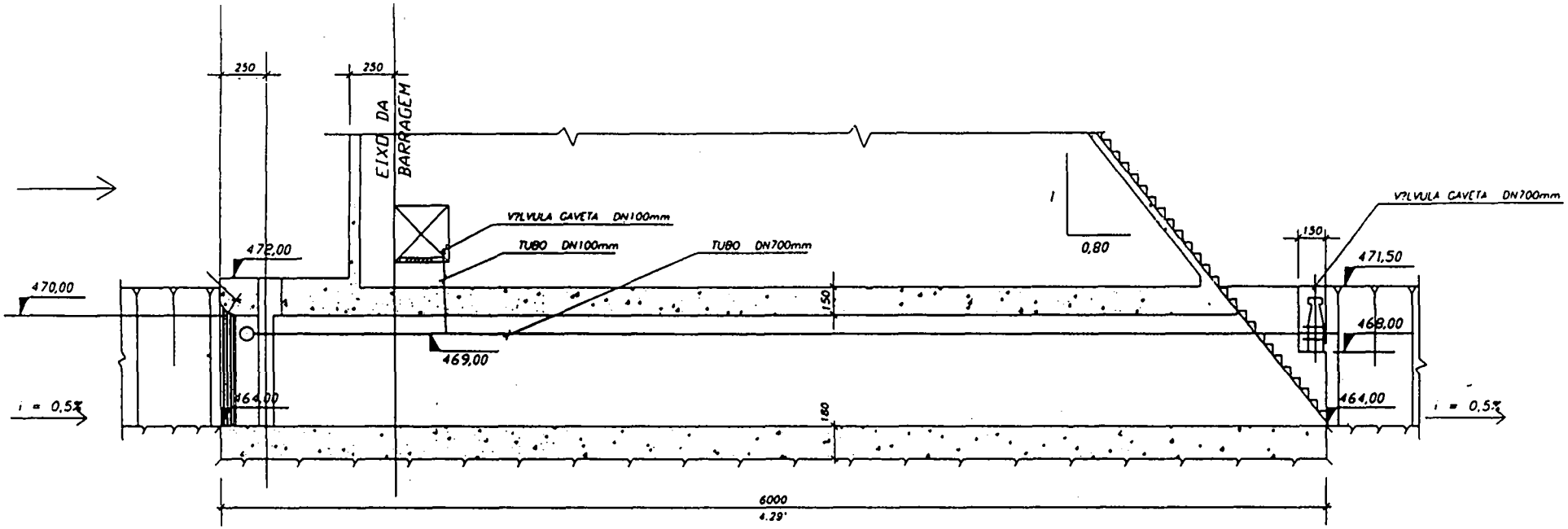


Figure 3. Location of Pilar Dam with respect to Ponte Nova city.



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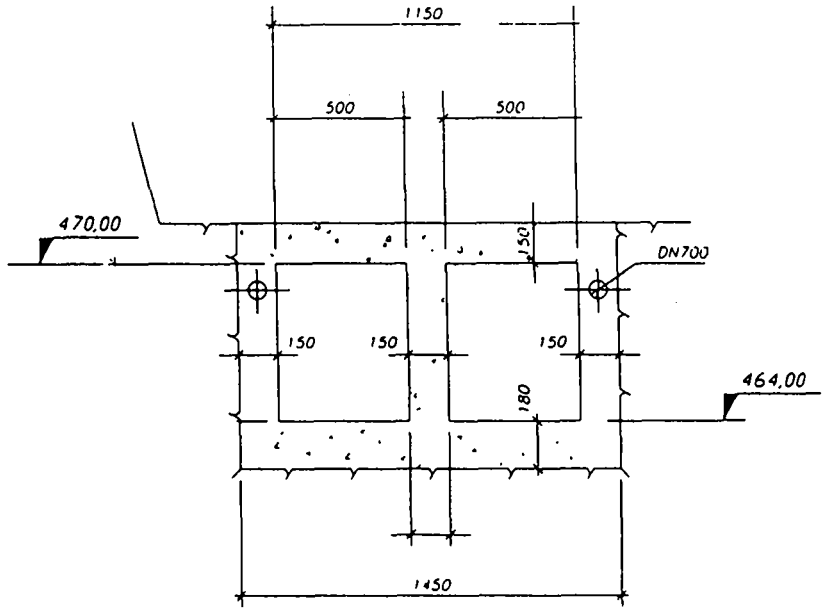


Figure 4. Pilar Dam diversion culverts.

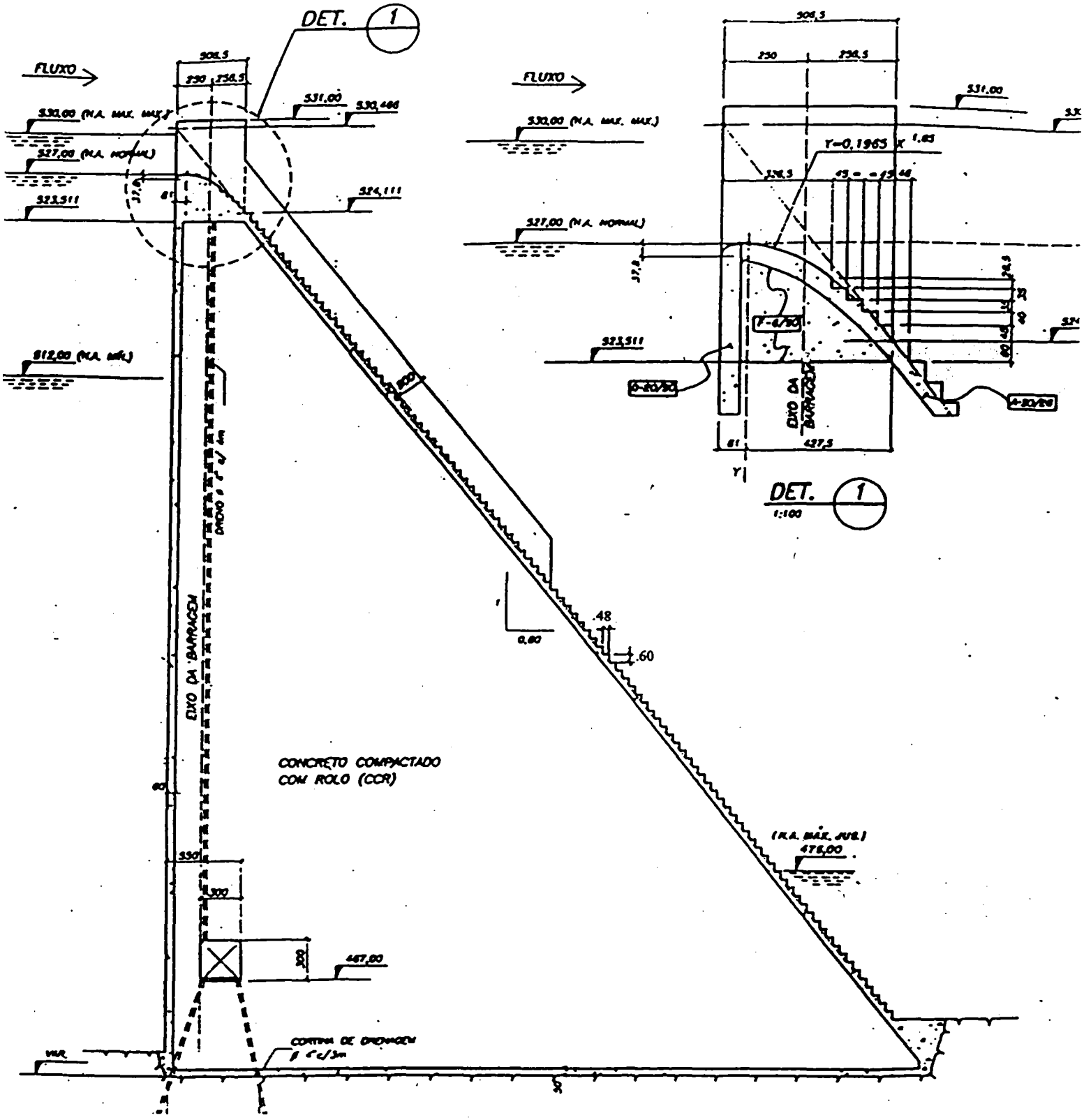


Figure 5. Stepped spillway section.

Figure 6. View of water surface profiles marked on Plexiglass training walls.

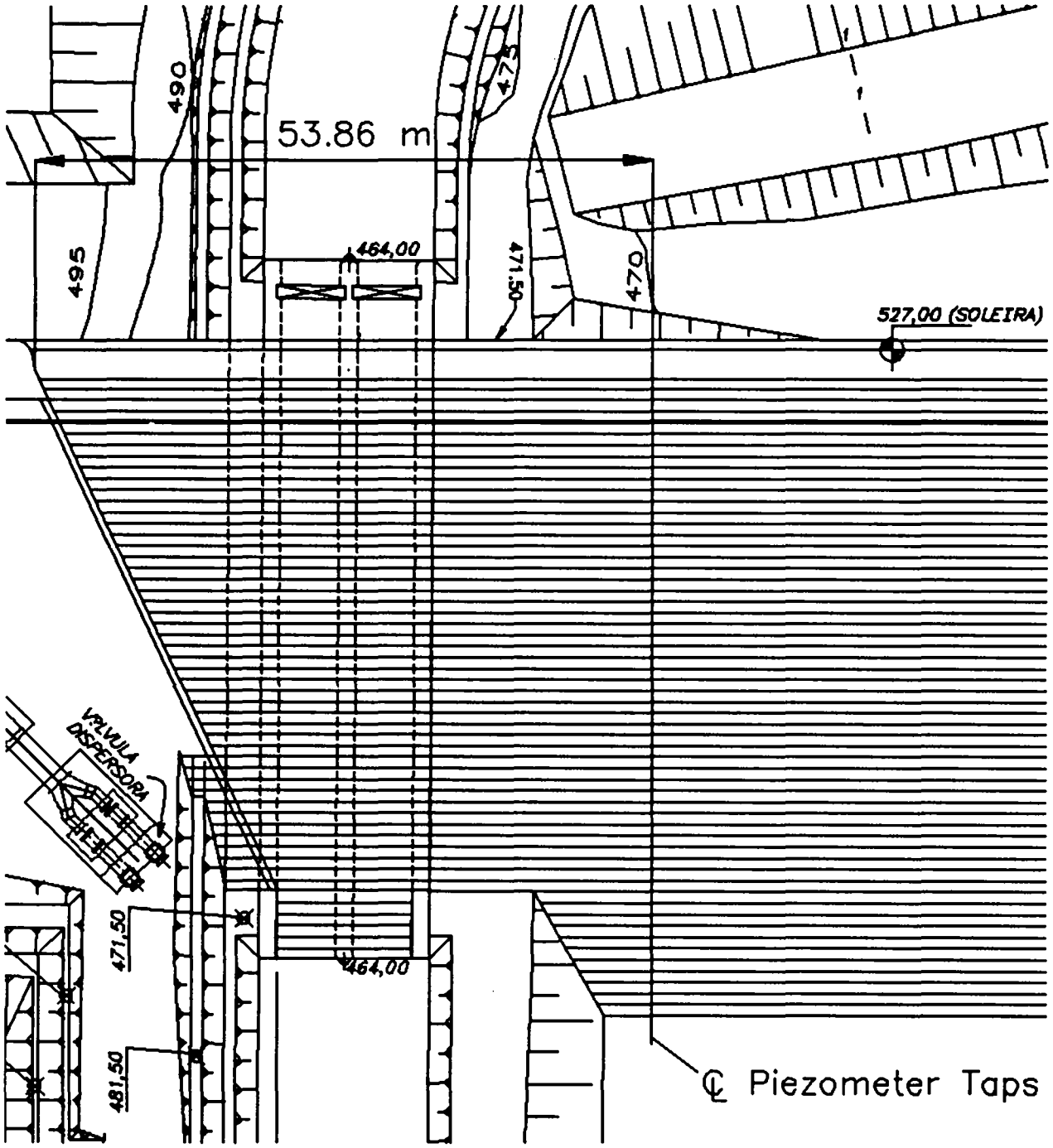


Figure 7. Piezometer tap locations on spillway.

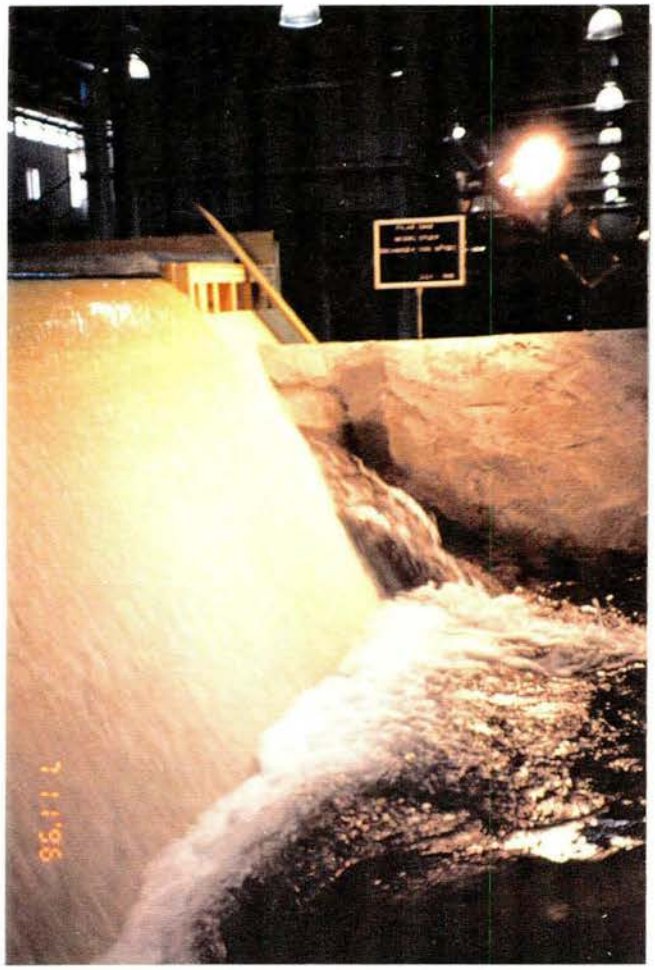


Figure 8. Original spillway design at 1000 m^3/s. (Note impact of flow on left abutment)

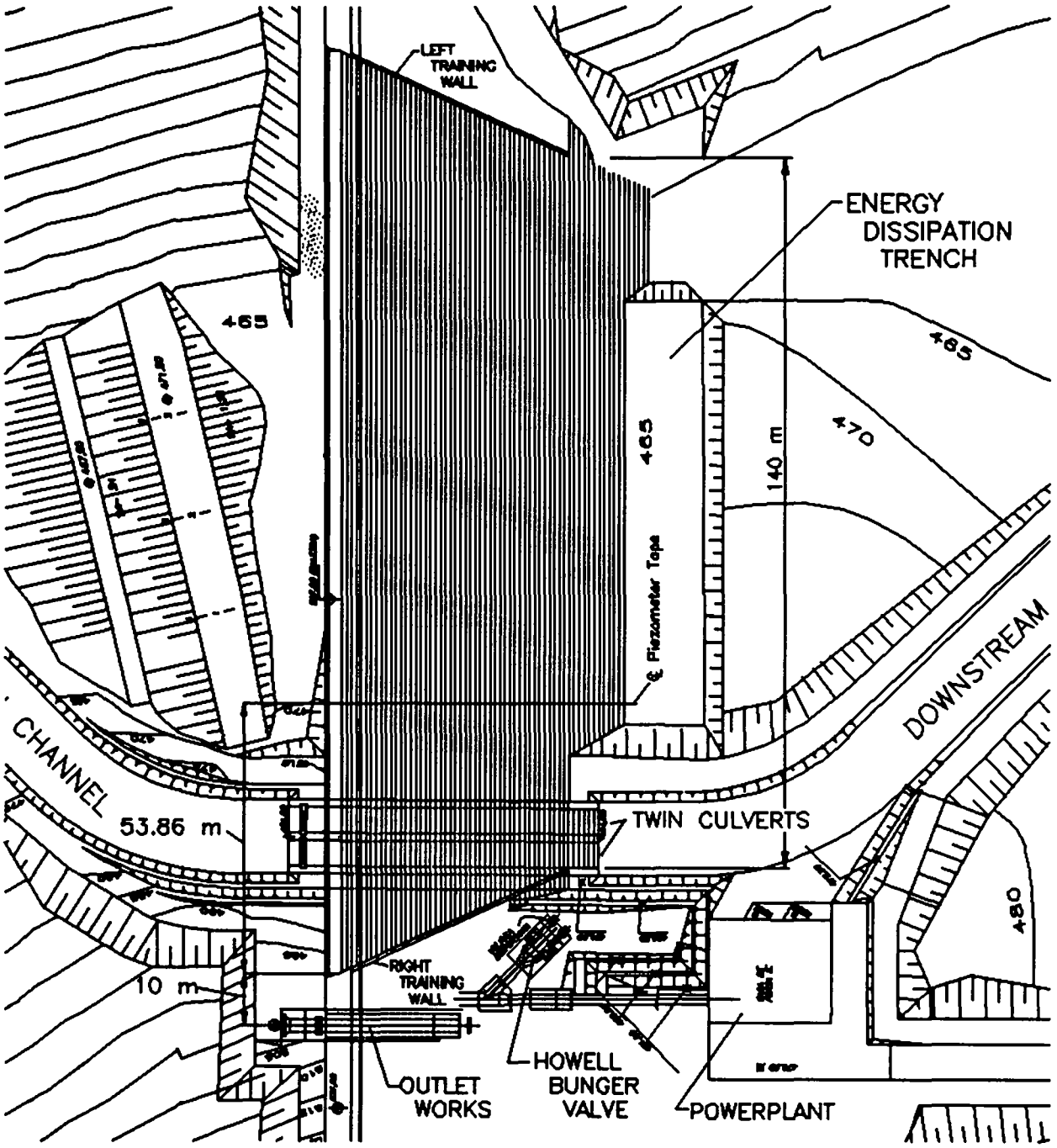
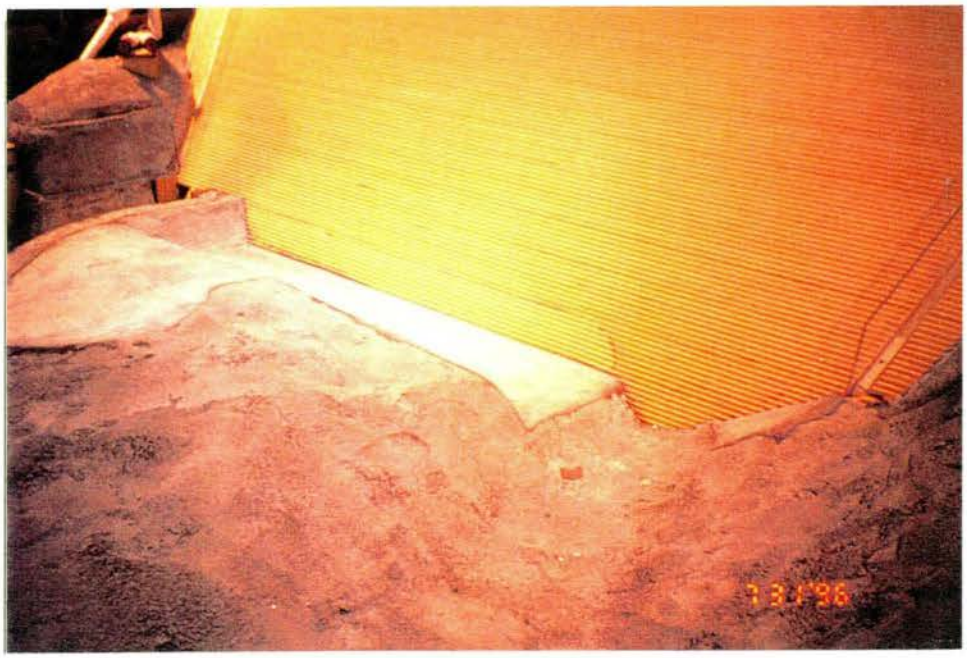
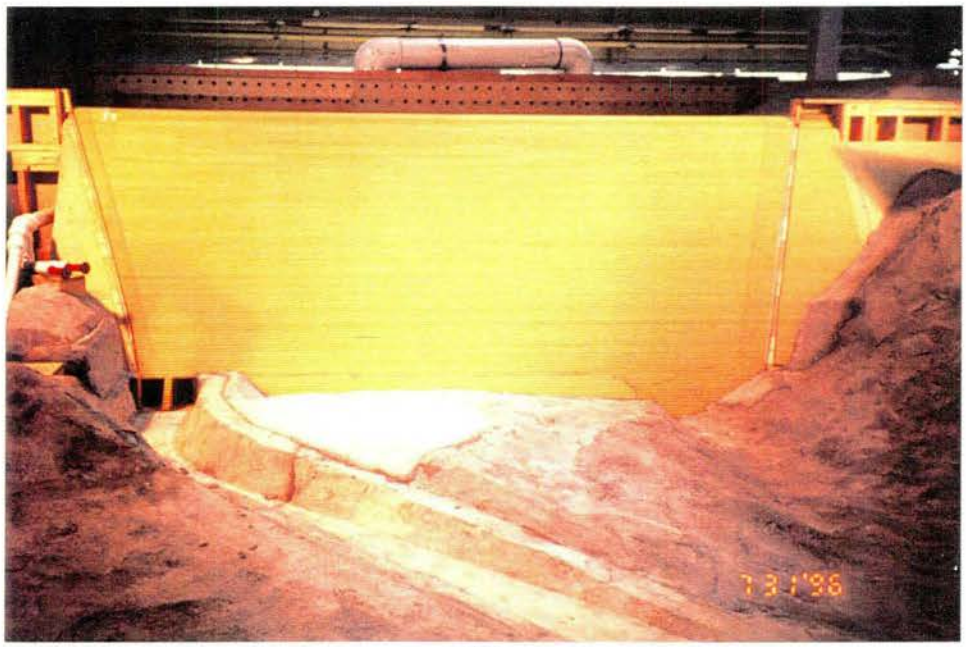


Figure 9. Plan view of final modified spillway.

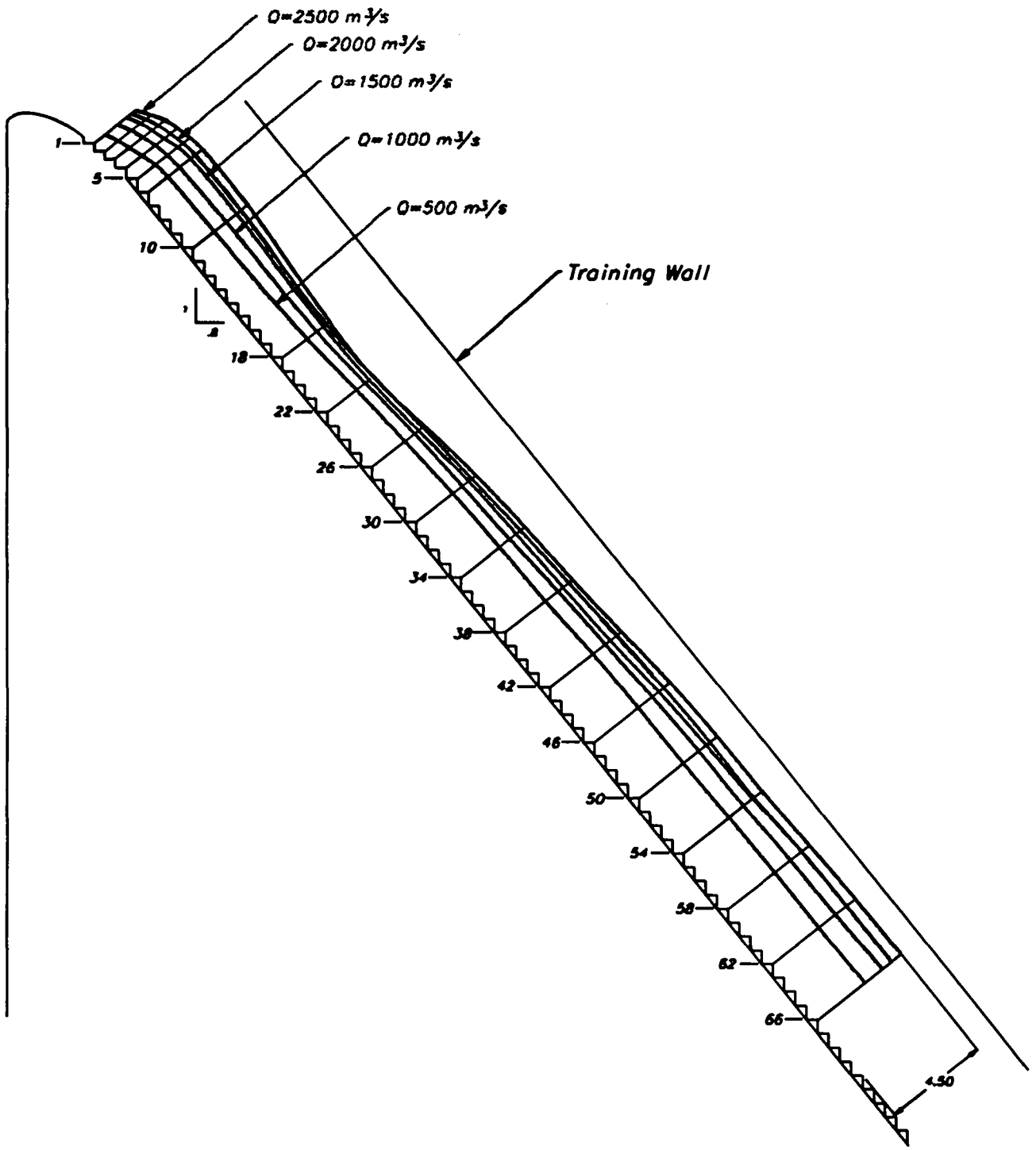


(a)



(b)

Figure 10. Final spillway design showing a) the excavated trench, and b) modified crest and training walls.



'igure 11. Converging spillway training wall water surface profiles.

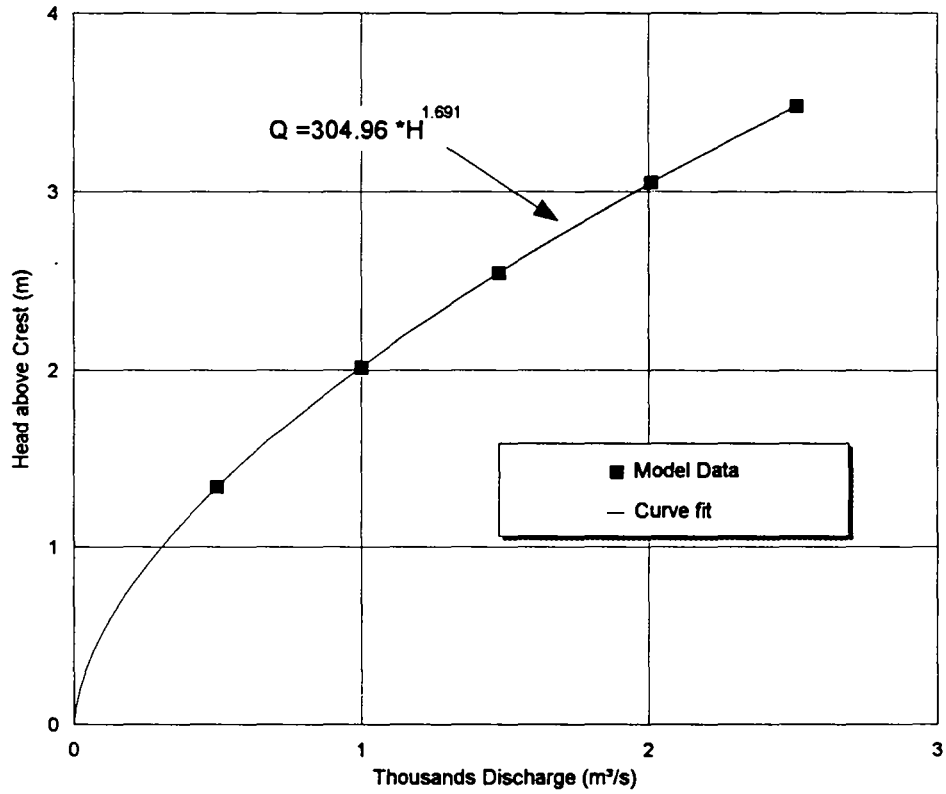


Figure 12. Discharge rating curve for spillway flows.

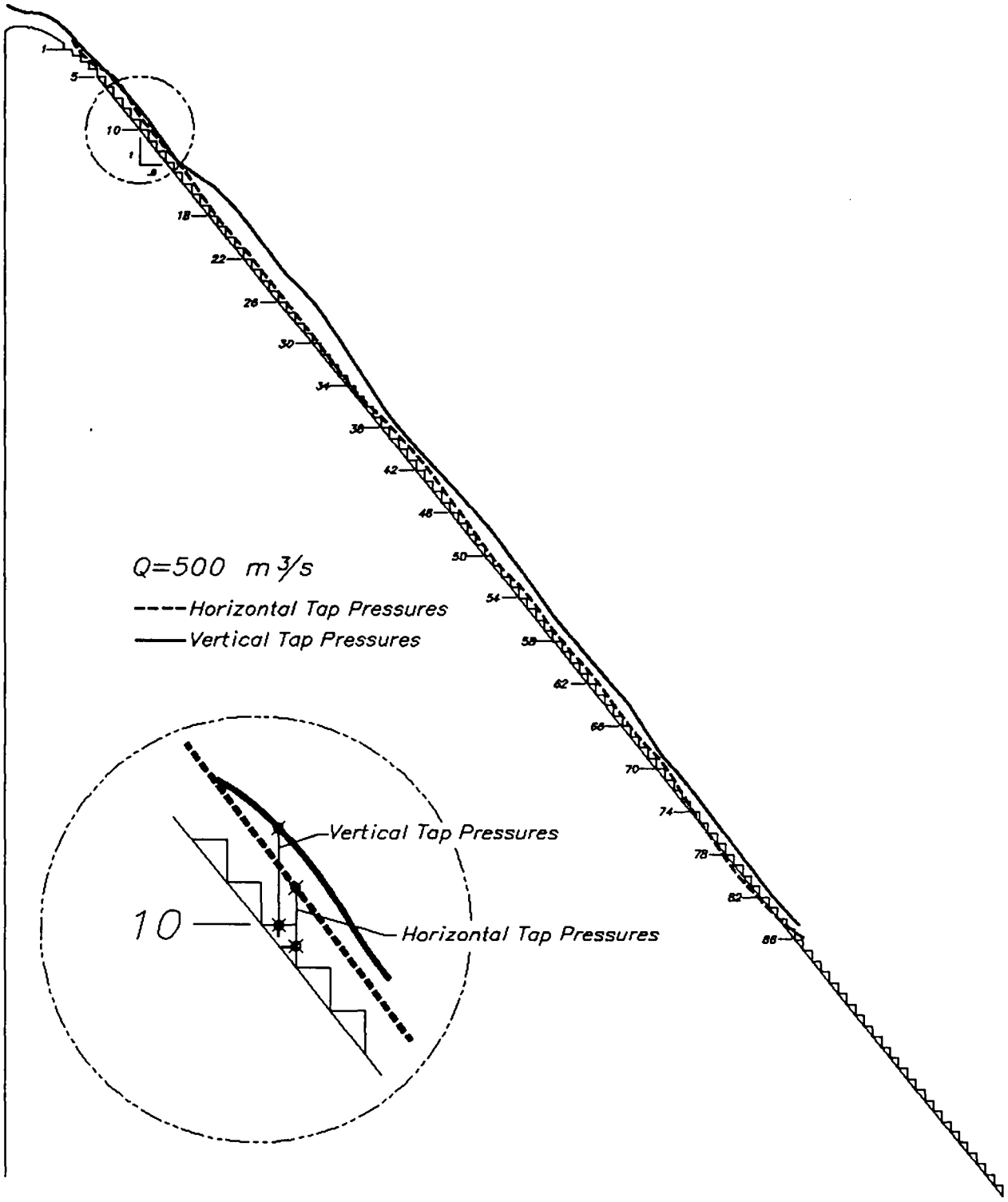


Figure 13. Spillway pressures measured at 500 m^3/s.

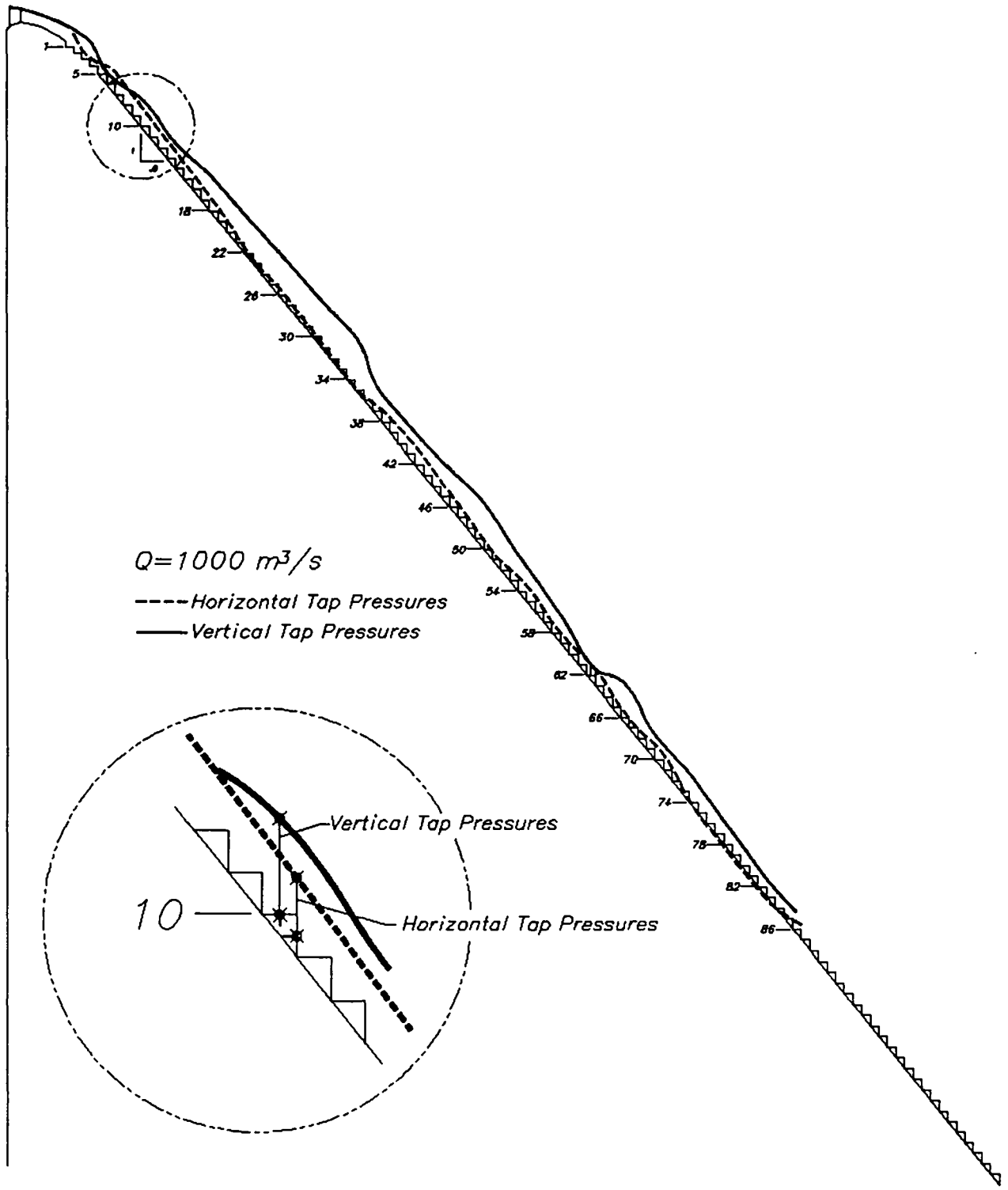


Figure 14. Spillway pressures measured at 1000 m^3/s.

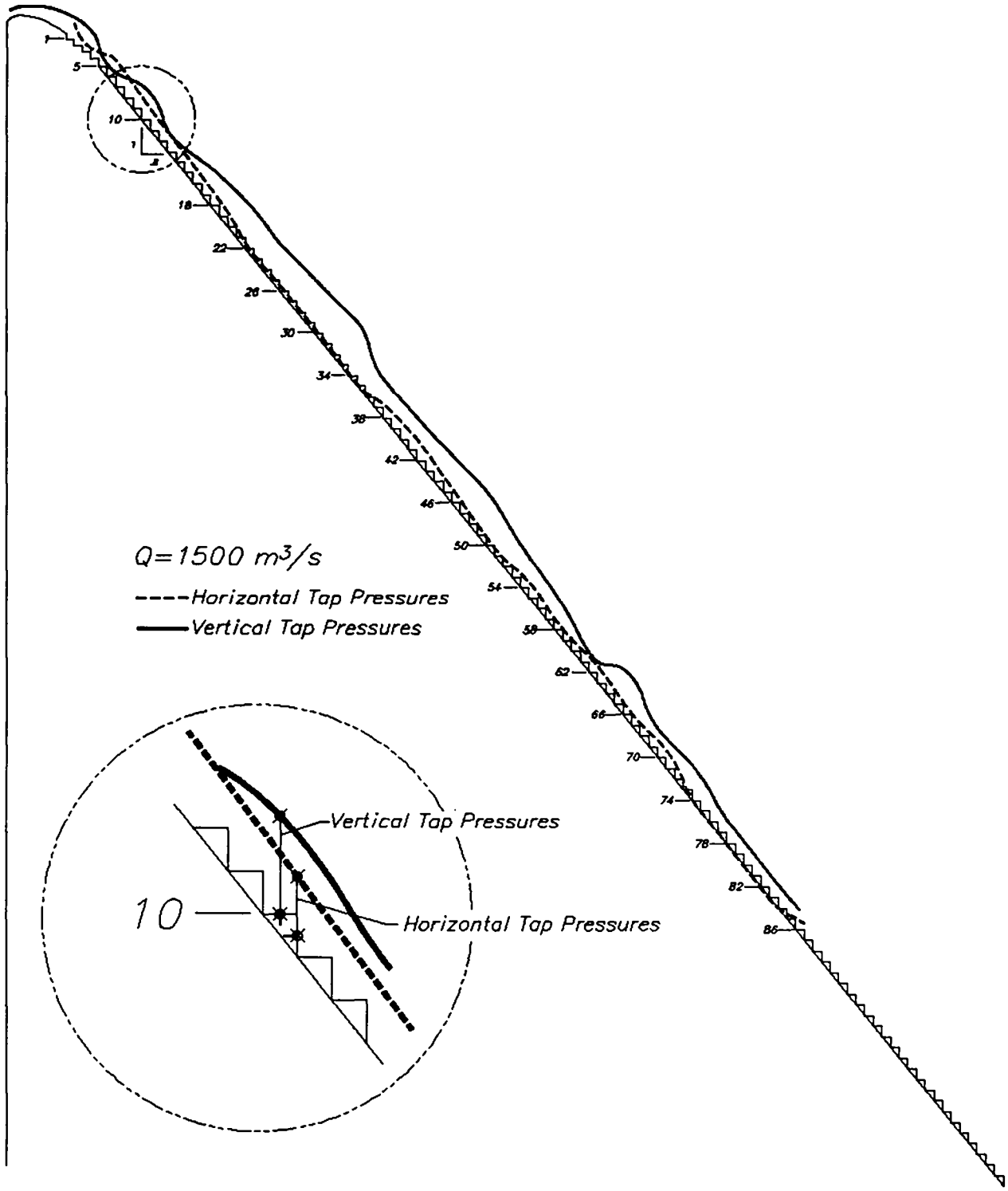


Figure 15. Spillway pressures measured at 1500 m^3/s.

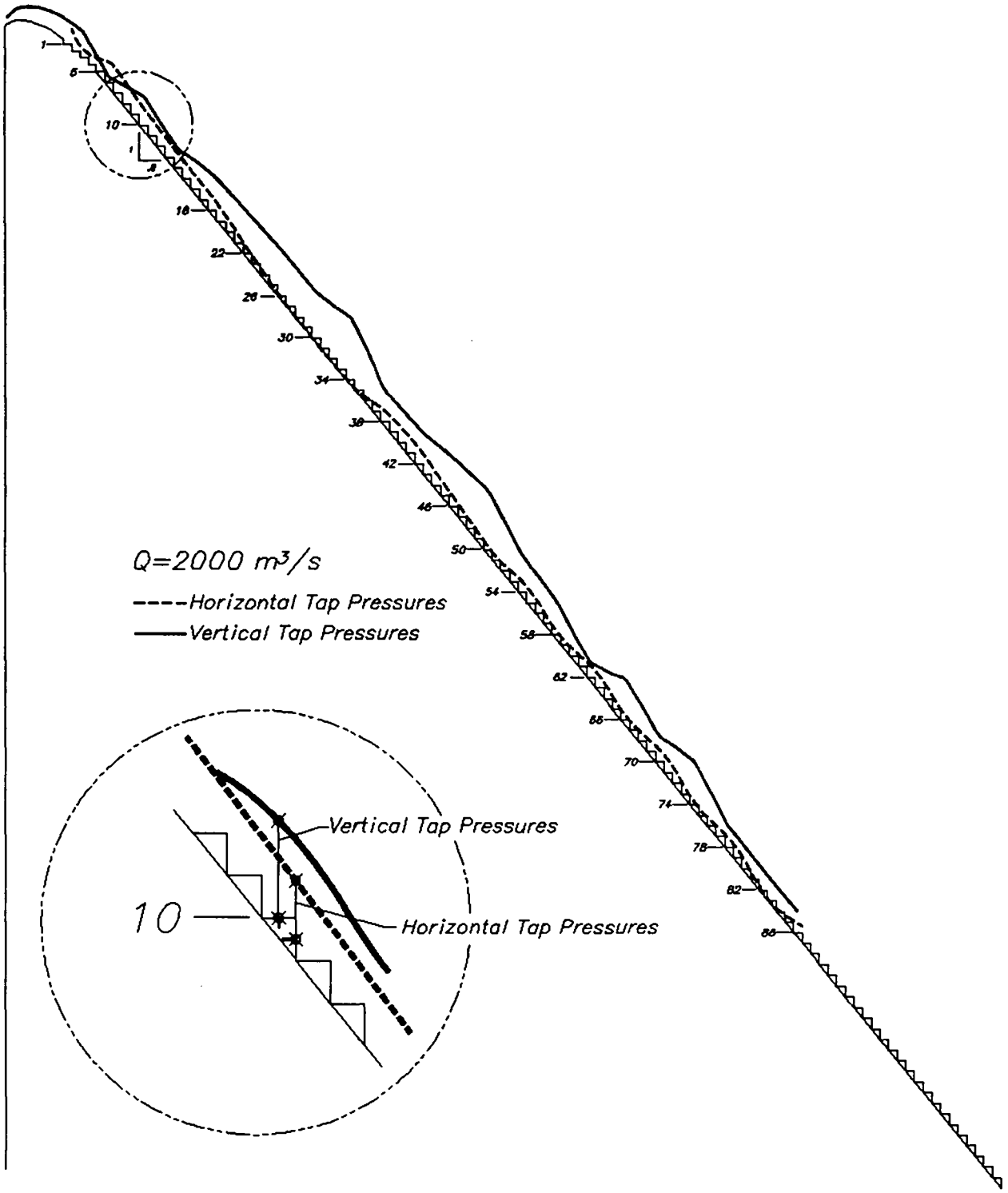


Figure 16. Spillway pressures measured at 2000 m^3/s.

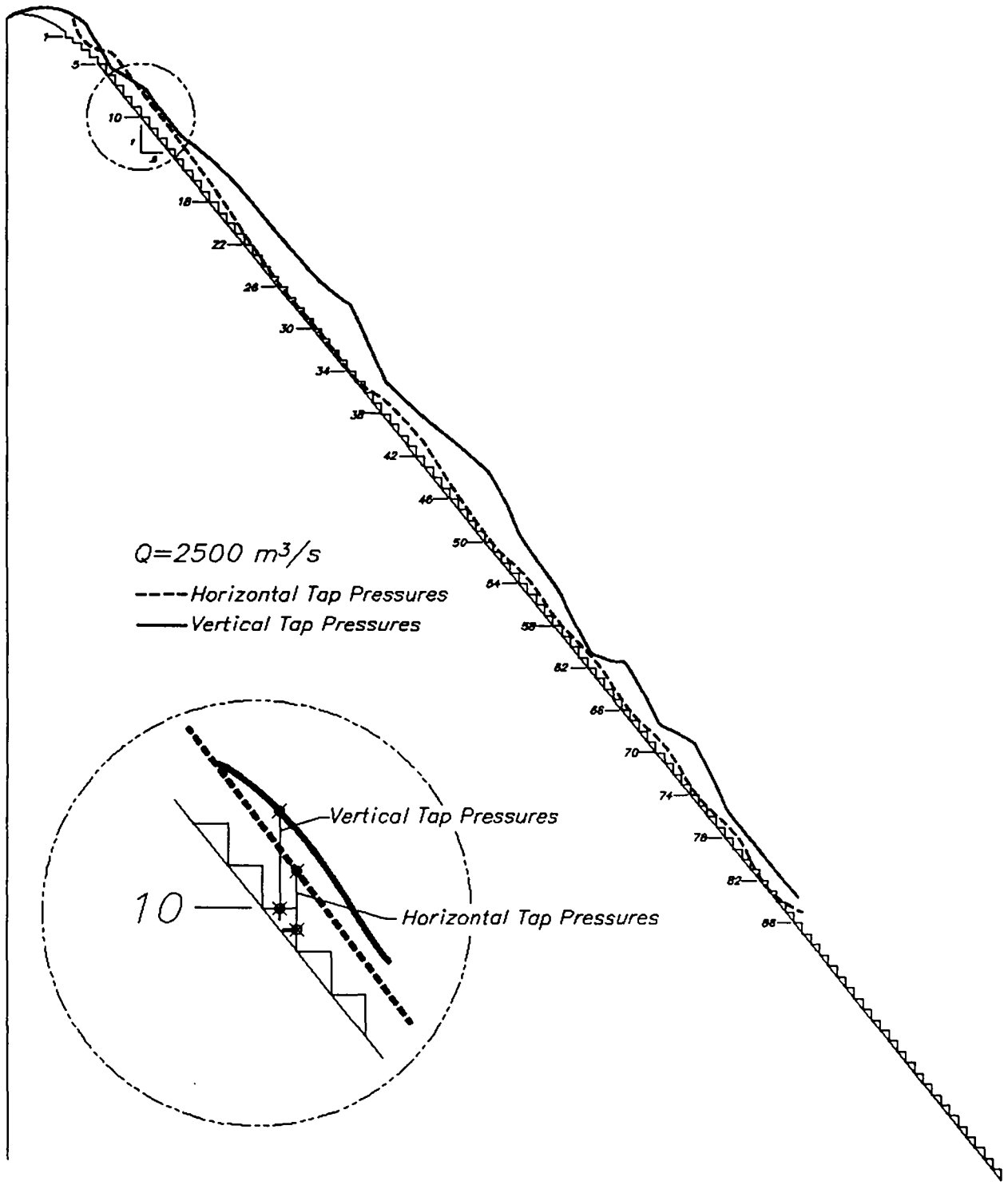


Figure 17. Spillway pressures measured at 2500 m^3/s.

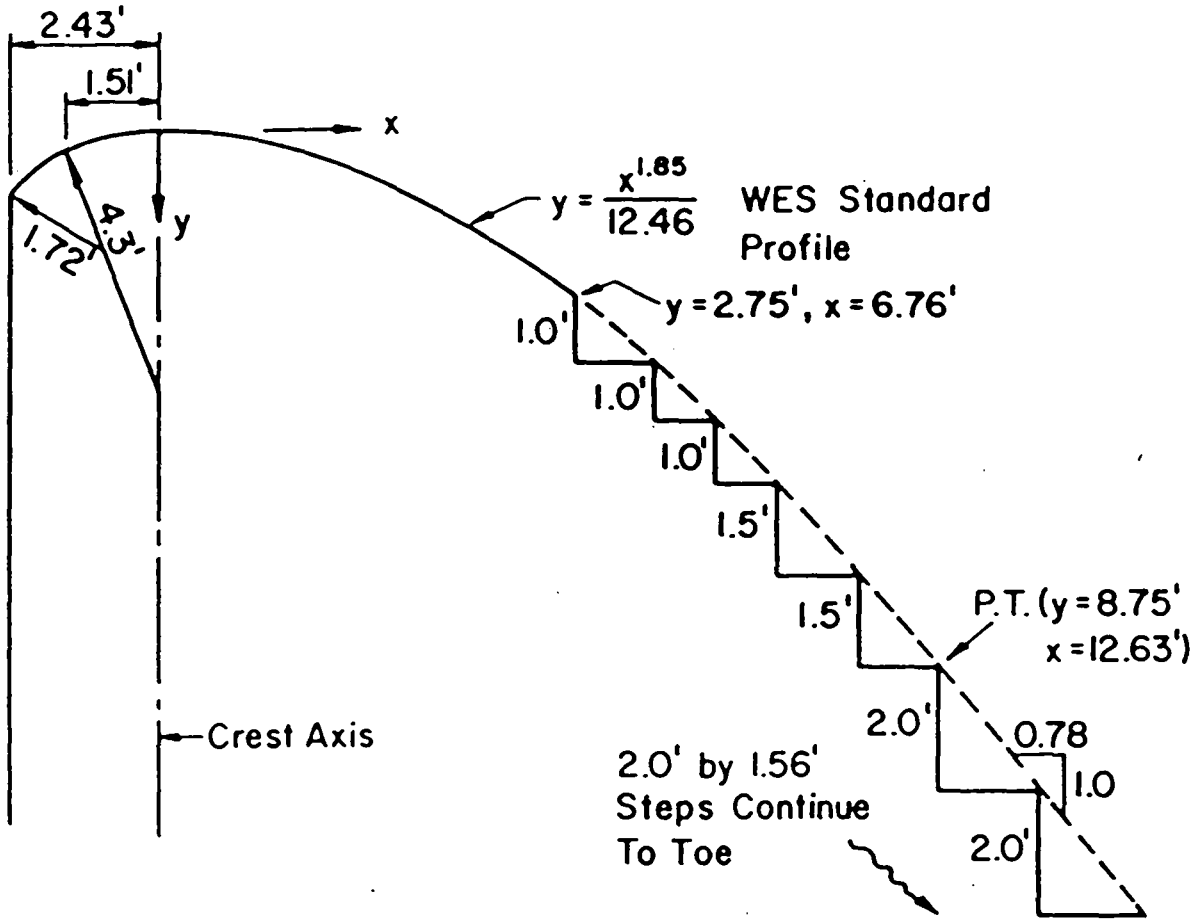
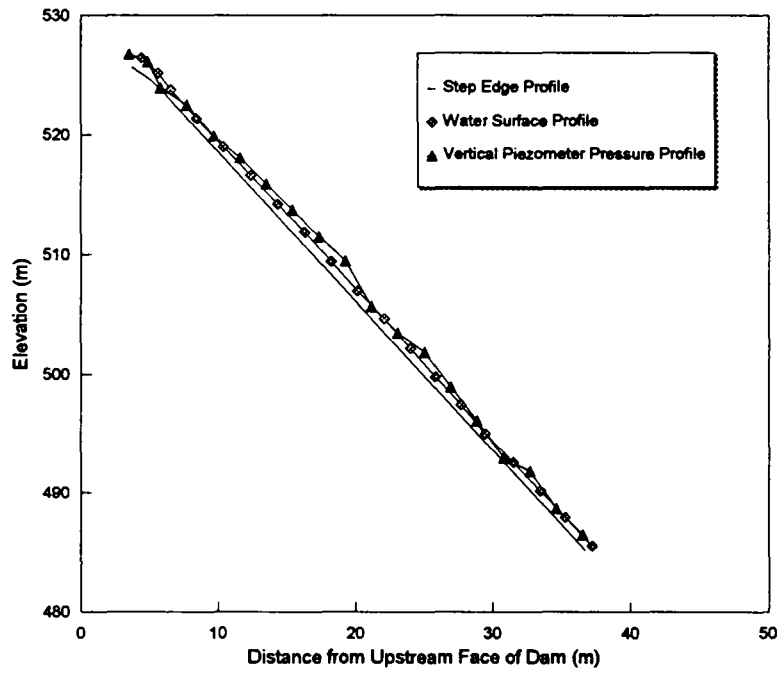
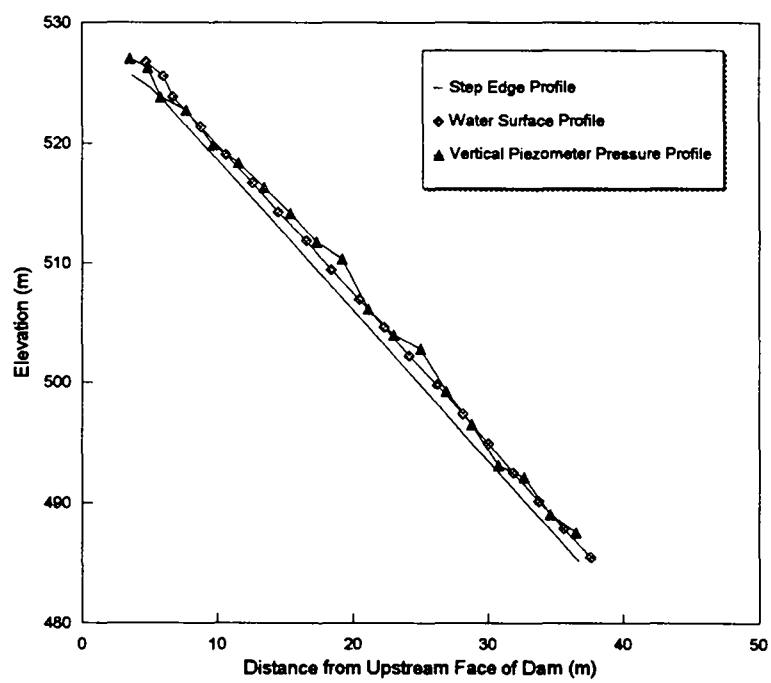


Figure 18. WES (Waterway Experiment Station, Corps of Engineers) standard profile and stepped profile, upper portion of spillway (Monksville Dam spillway).



a)



b)

Figure 19. Water surface profiles over the center of the spillway for a) 1000 m³/s and b) 2000 m³/s.

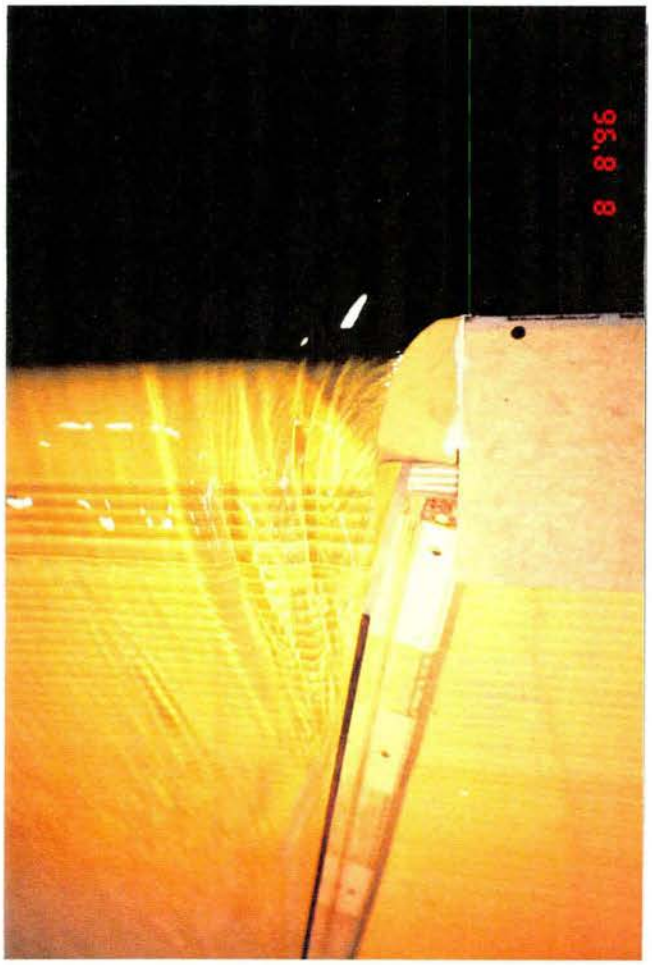


Figure 20. View of flow around spillway piers at 2000 m^3/s.

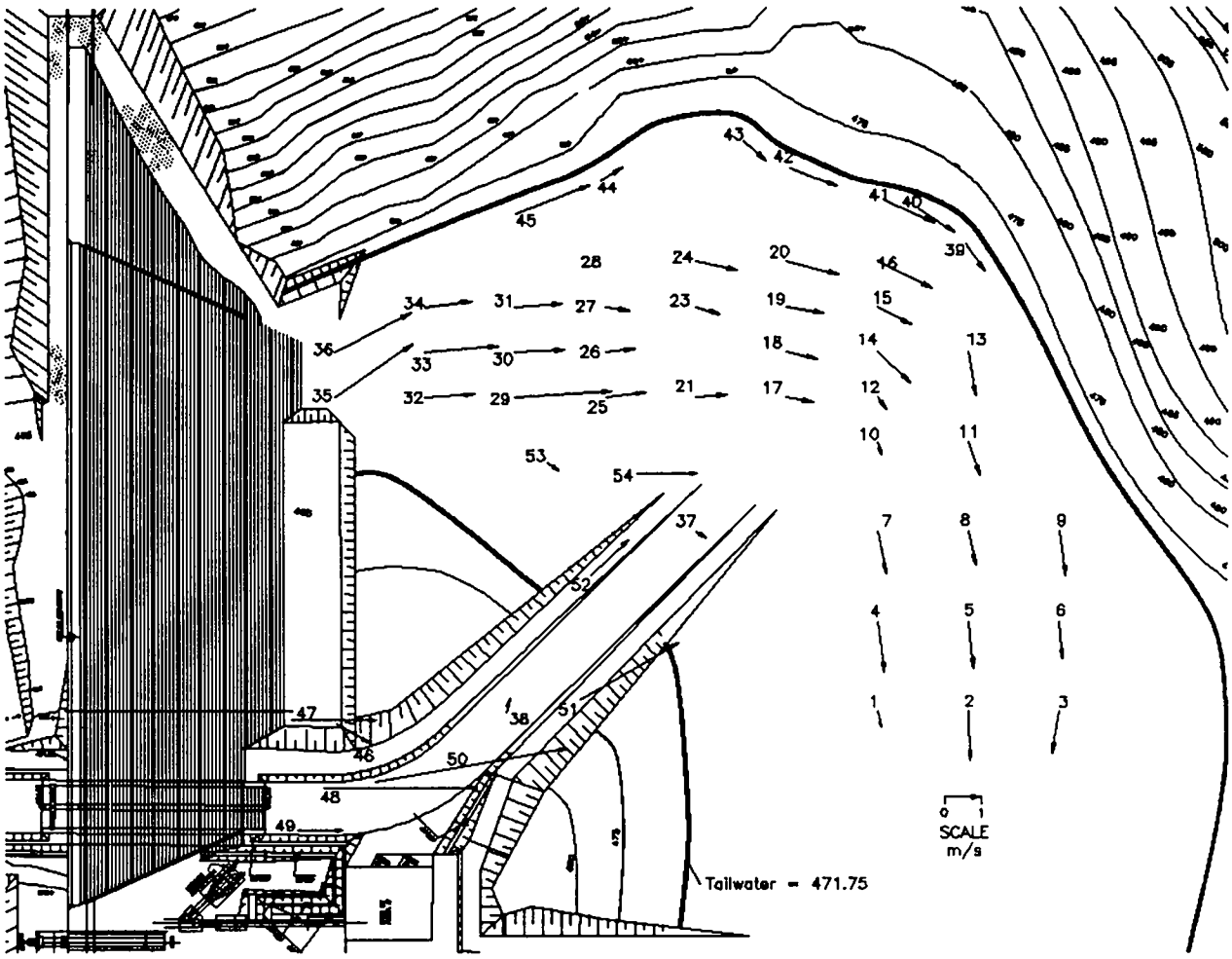


Figure 21. River and diversion channel velocities measured at a spillway discharge of 1000 m^3/s (length of arrows indicate relative magnitude and direction of velocity at each numbered grid point).

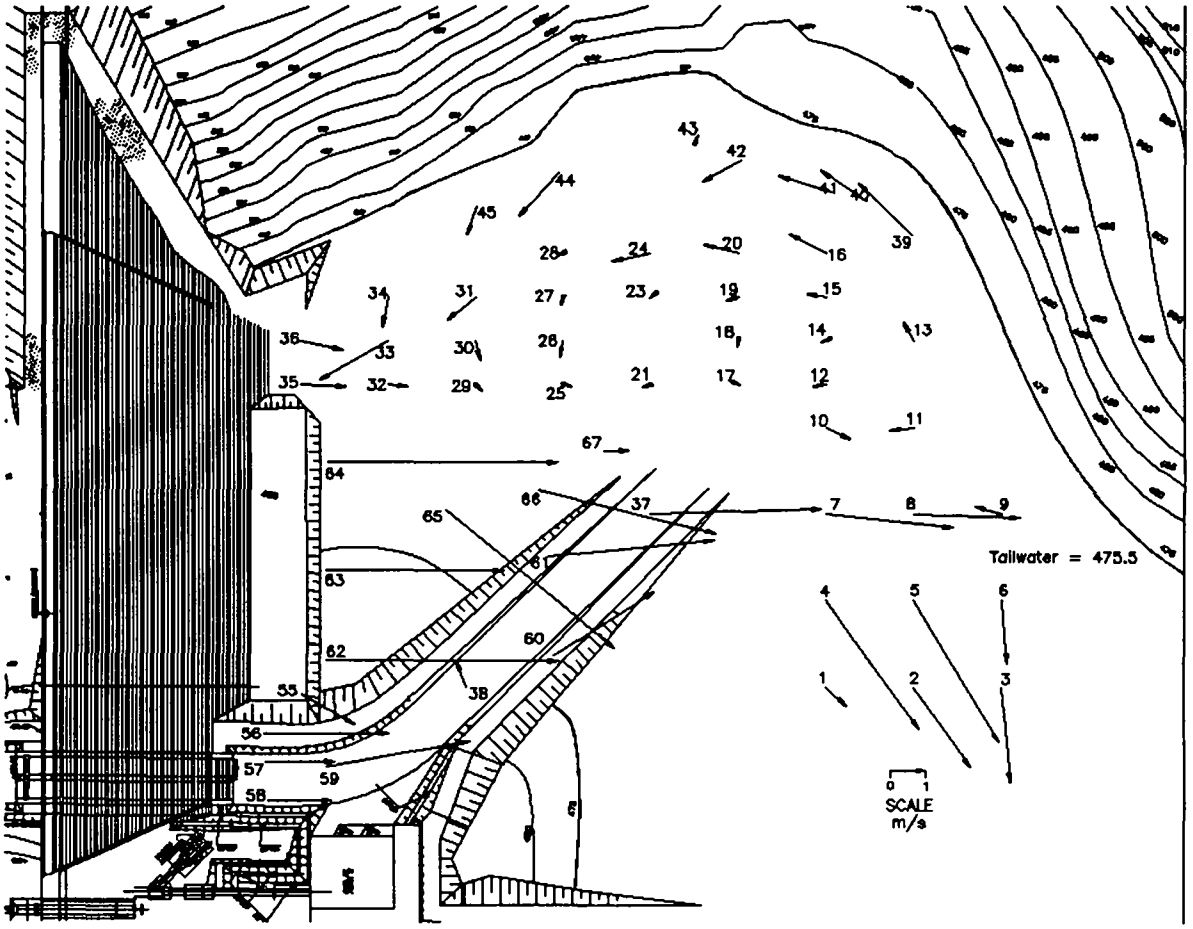


Figure 22. River and diversion channel velocities measured at a spillway discharge of 2000 m^3/s (length of arrows indicate relative magnitude and direction of velocities at each numbered grid point).

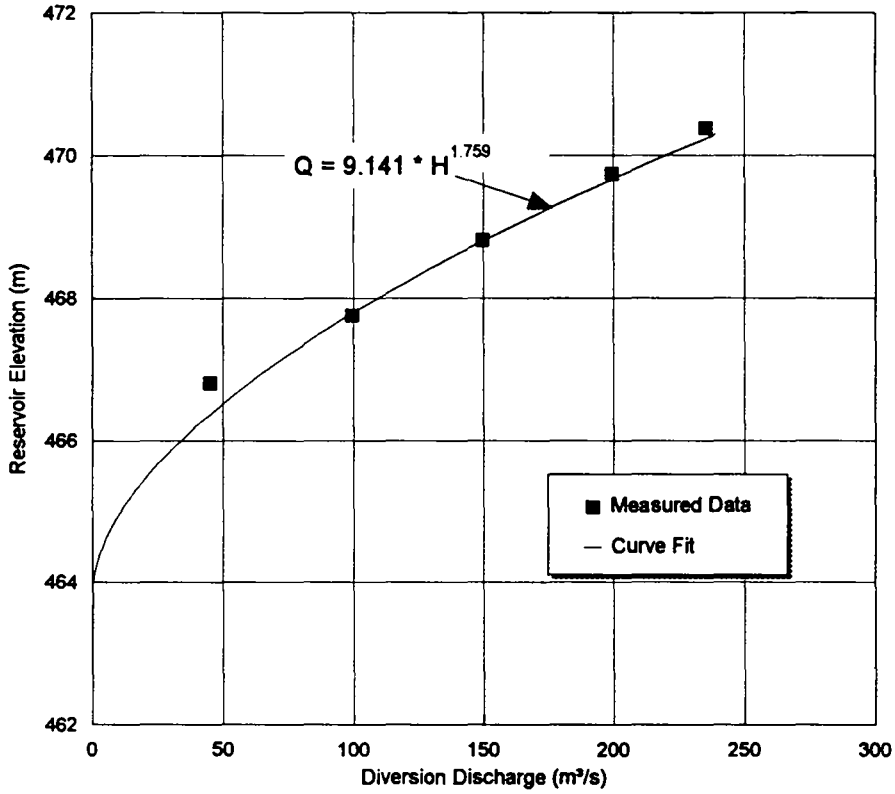


Figure 23. Discharge rating curve for diversion flows.

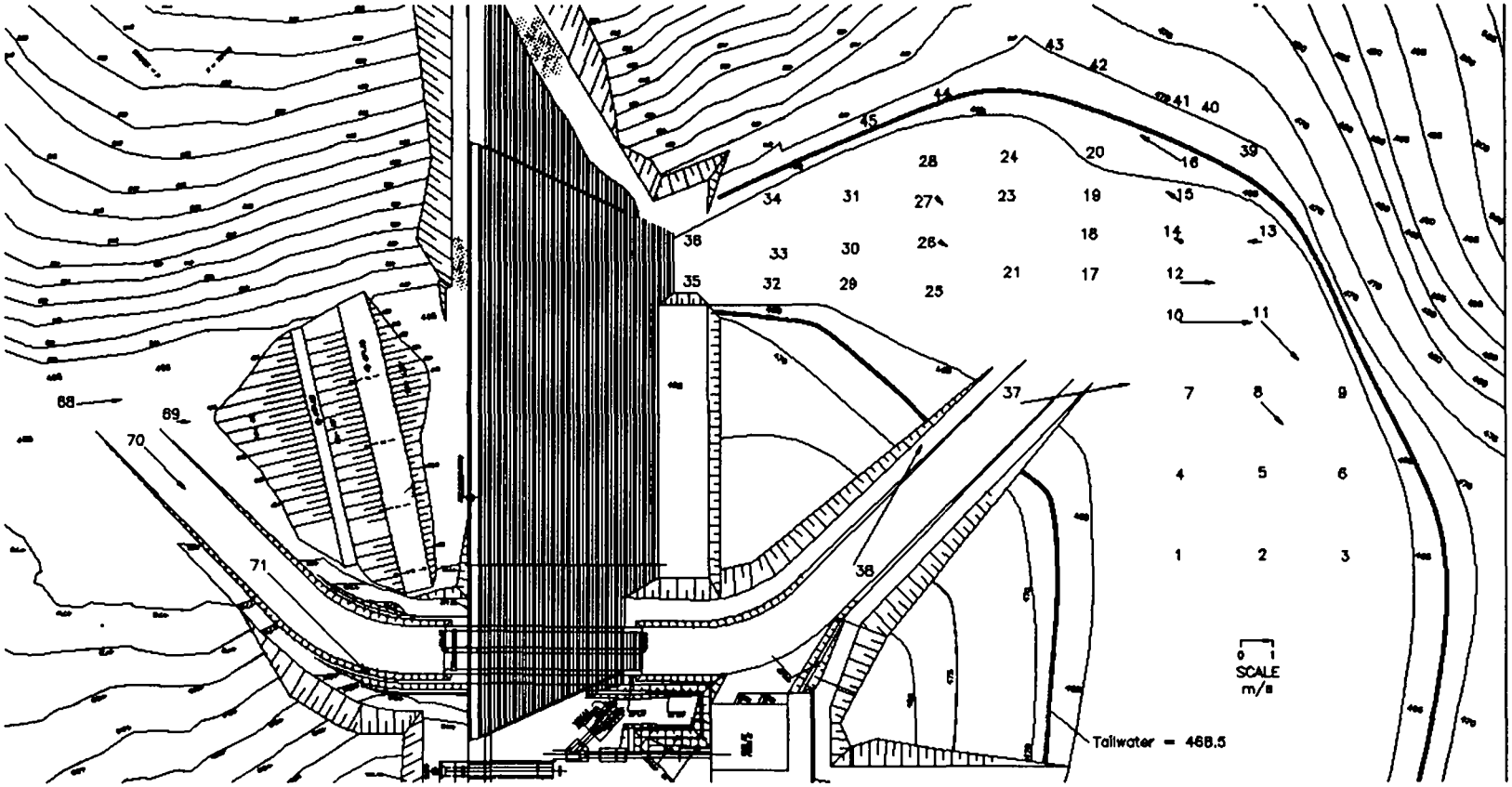
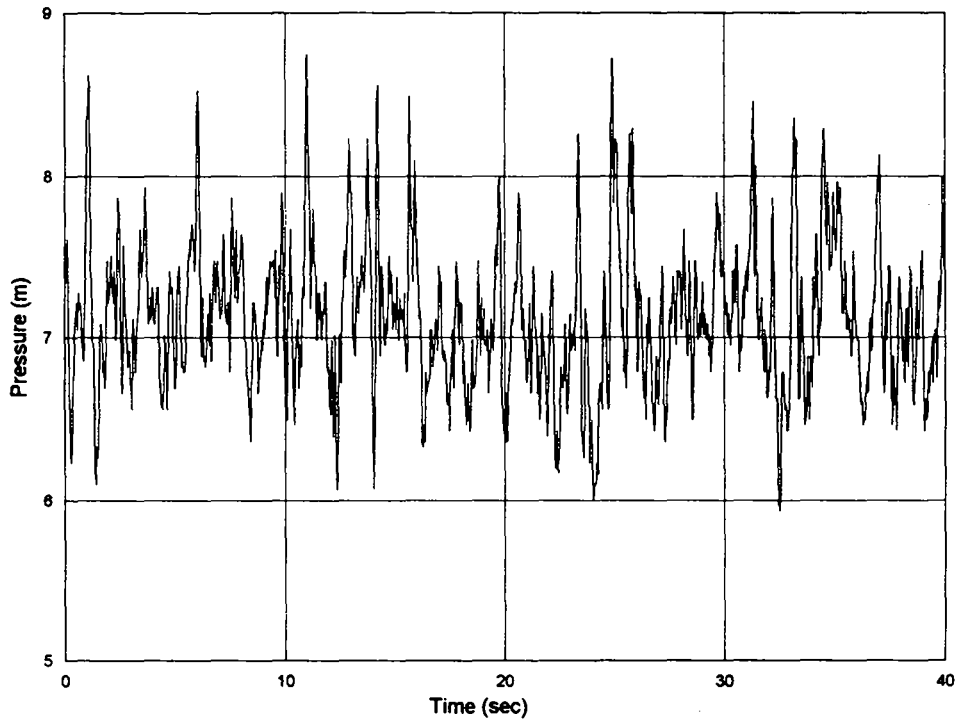


Figure 24. River and diversion channel velocities measured at a diversion discharge of 235 m^3/s.



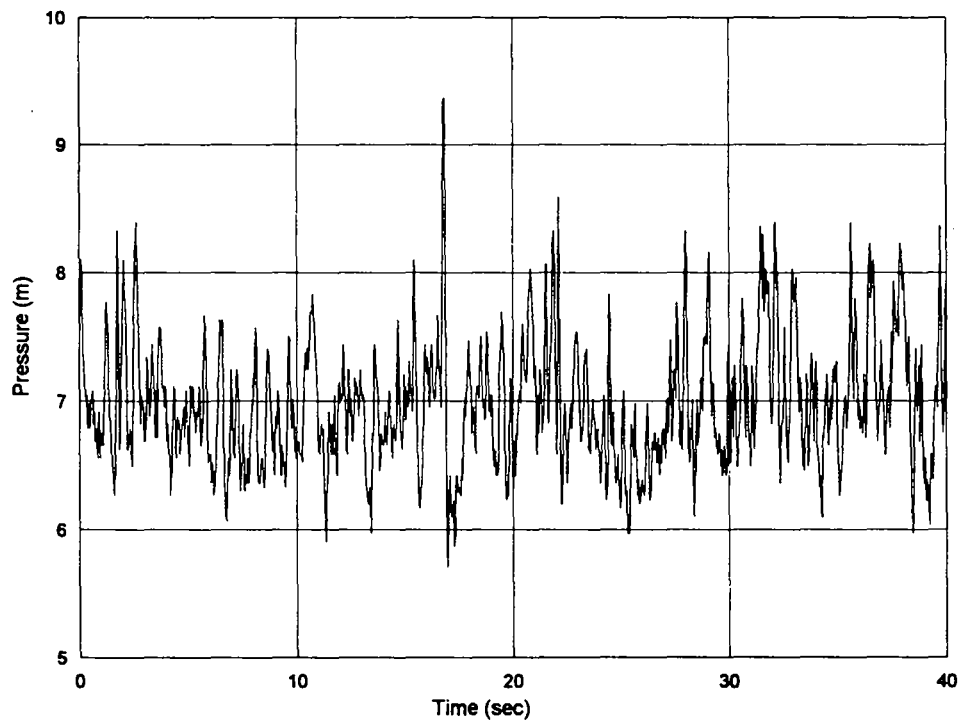
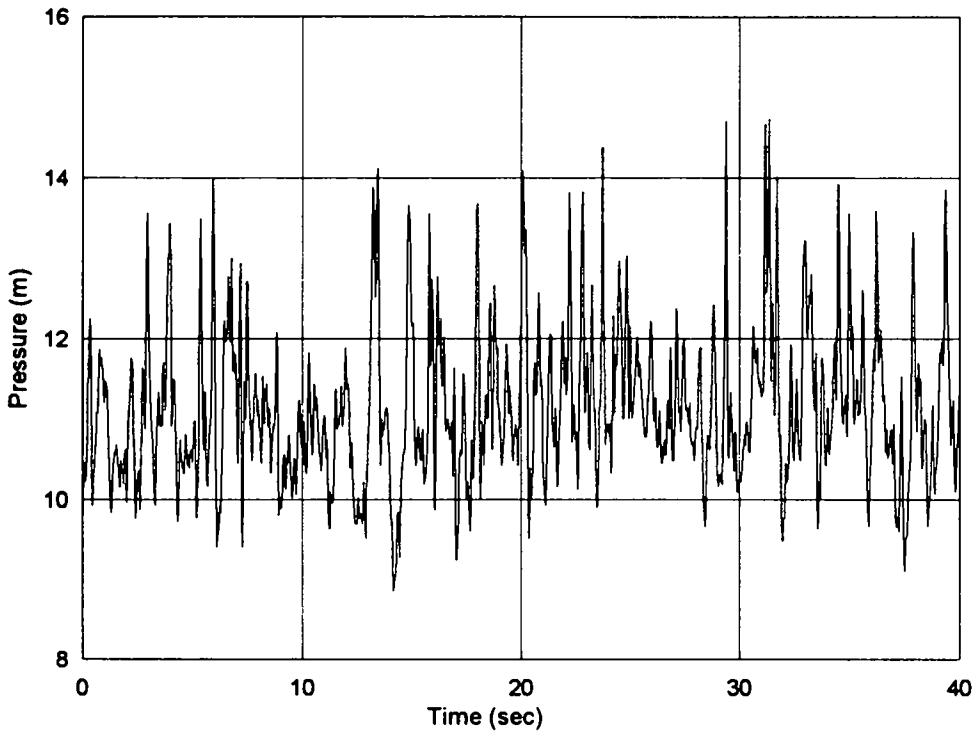


Figure 25. Pressure traces in 40 sec intervals for a flow of 1000 m^3/s (pressures are referenced to elevation 465 m).



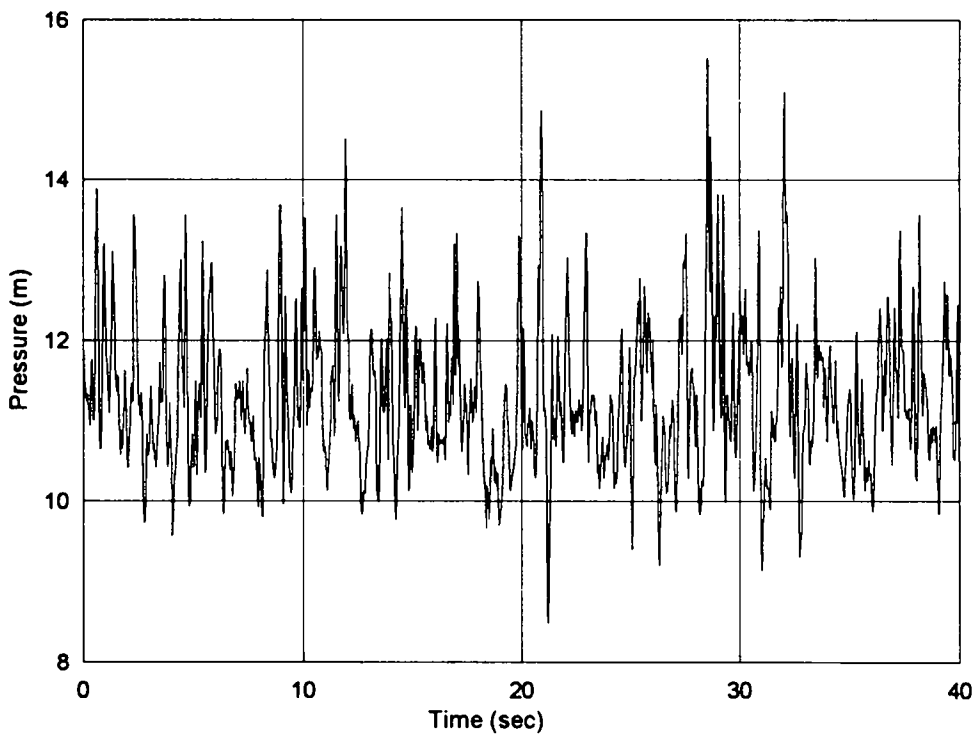
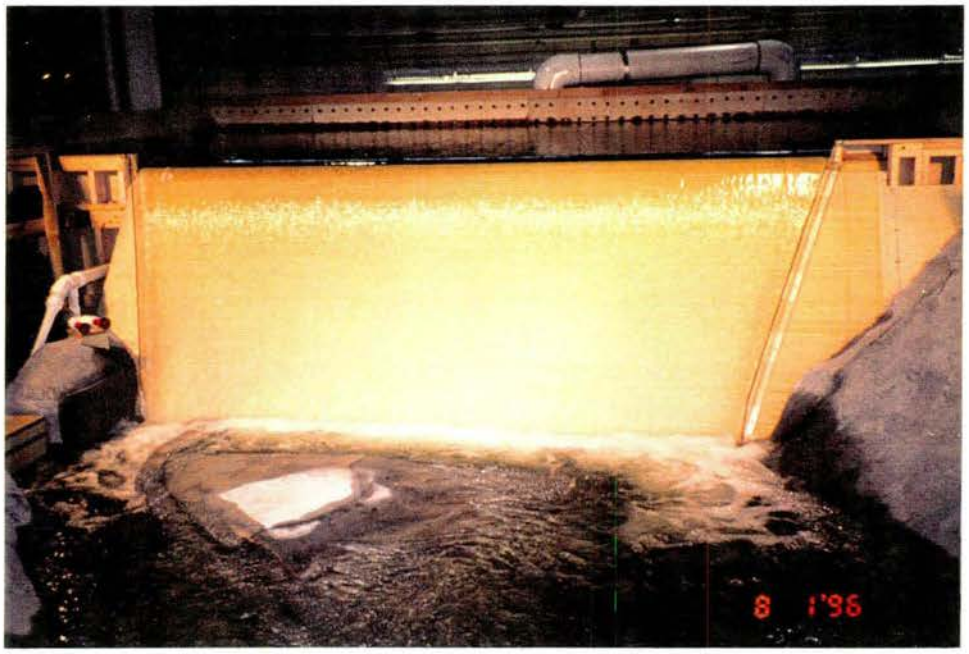


Figure 26. Pressure traces in 40 sec intervals for a flow of 2000 m^3/s (pressures are referenced to elevation 465 m).



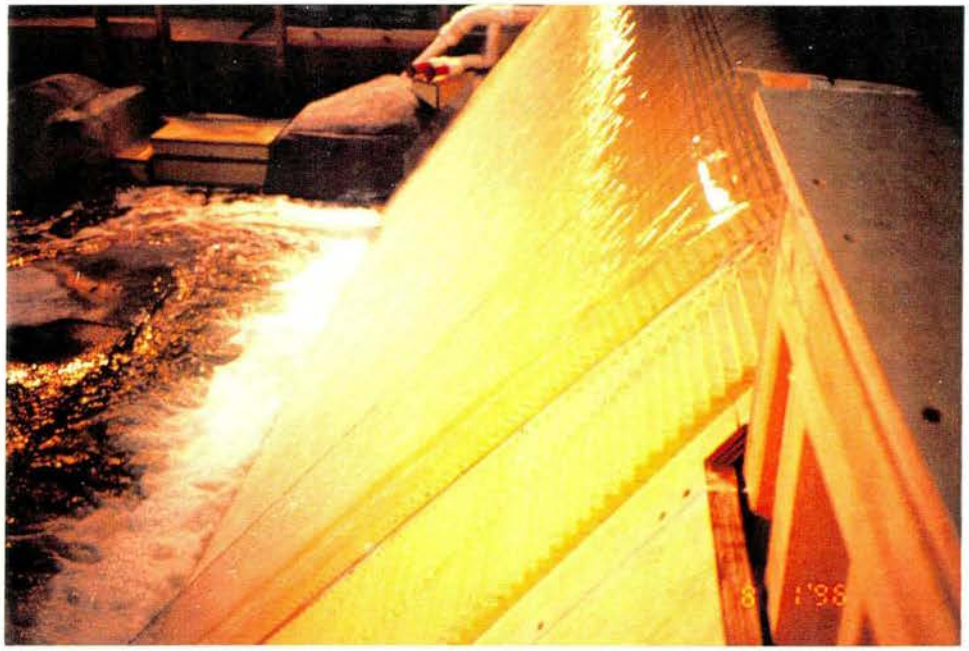


Figure 27. Final spillway design at 1000 m^3/s.

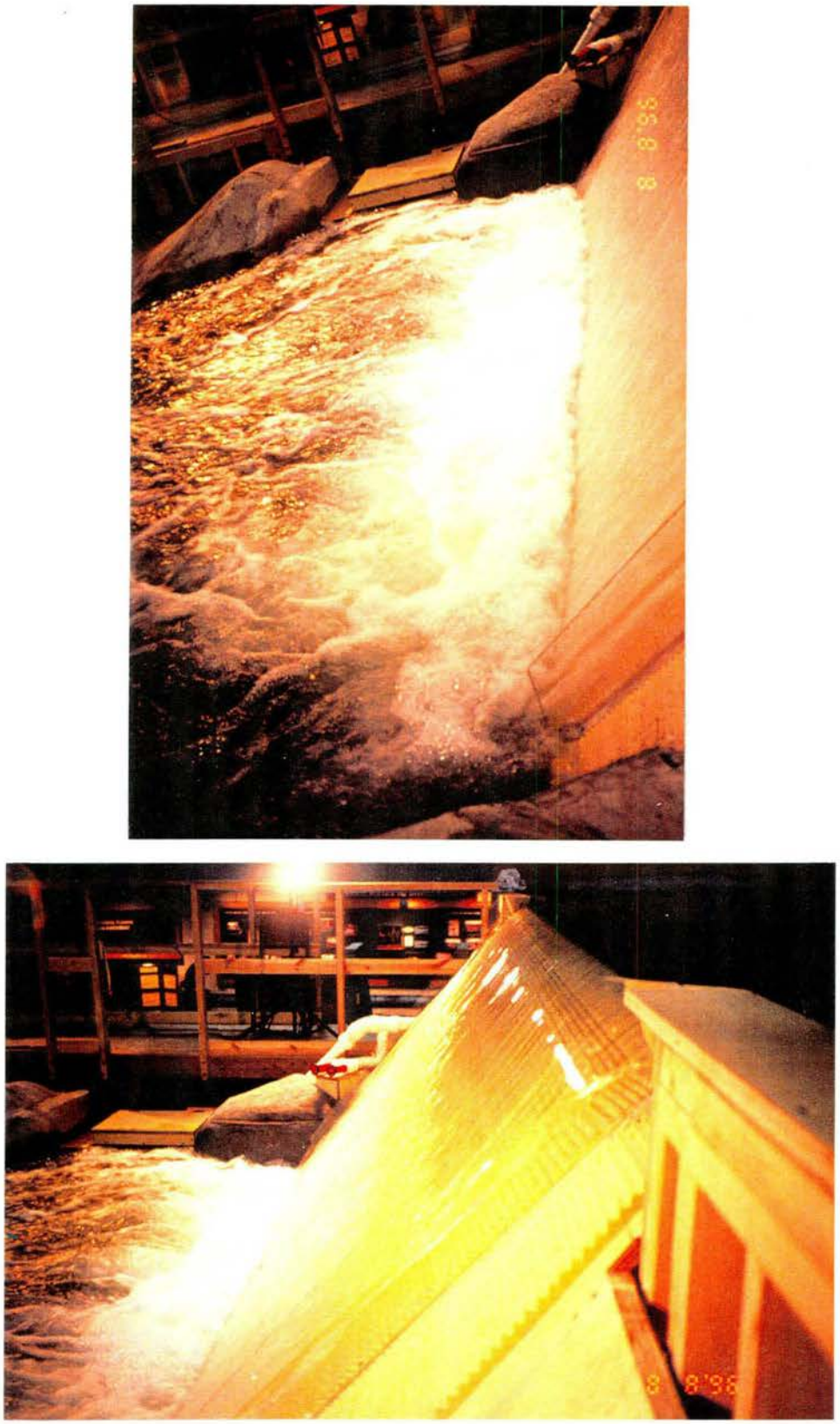


Figure 28. Final spillway design at 2000 m^3/s.

Figure 29. View of spillway trench filled with 1-m-diameter rock to elevation 467 m.

(a) (b)

Figure 30. Spillway trench filled with 1-m-diameter rock to elevation 467 m a) during a flow of 1000 m^3/s, and b) after a flow of 1000 m^3/s.

(a)

(b)

Figure 31. Spillway trench filled with 1-m-diameter rock to elevation 467 m a) during a flow of 2000 m^3/s, and b) after a flow of 2000 m^3/s. (Note pressure transducer mounting at base of spillway).

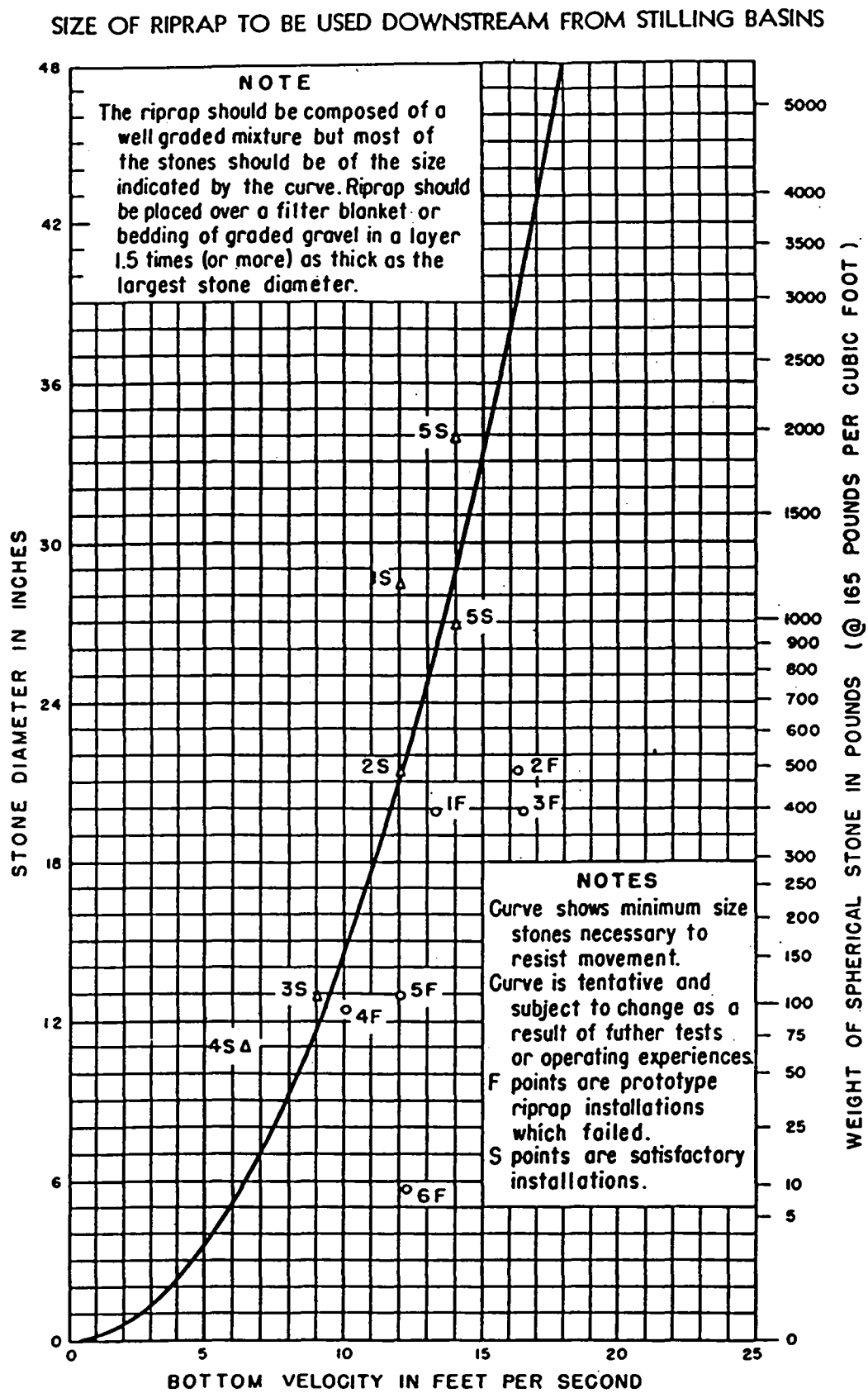


Figure 32. Curve showing minimum size stones necessary to resist movement.

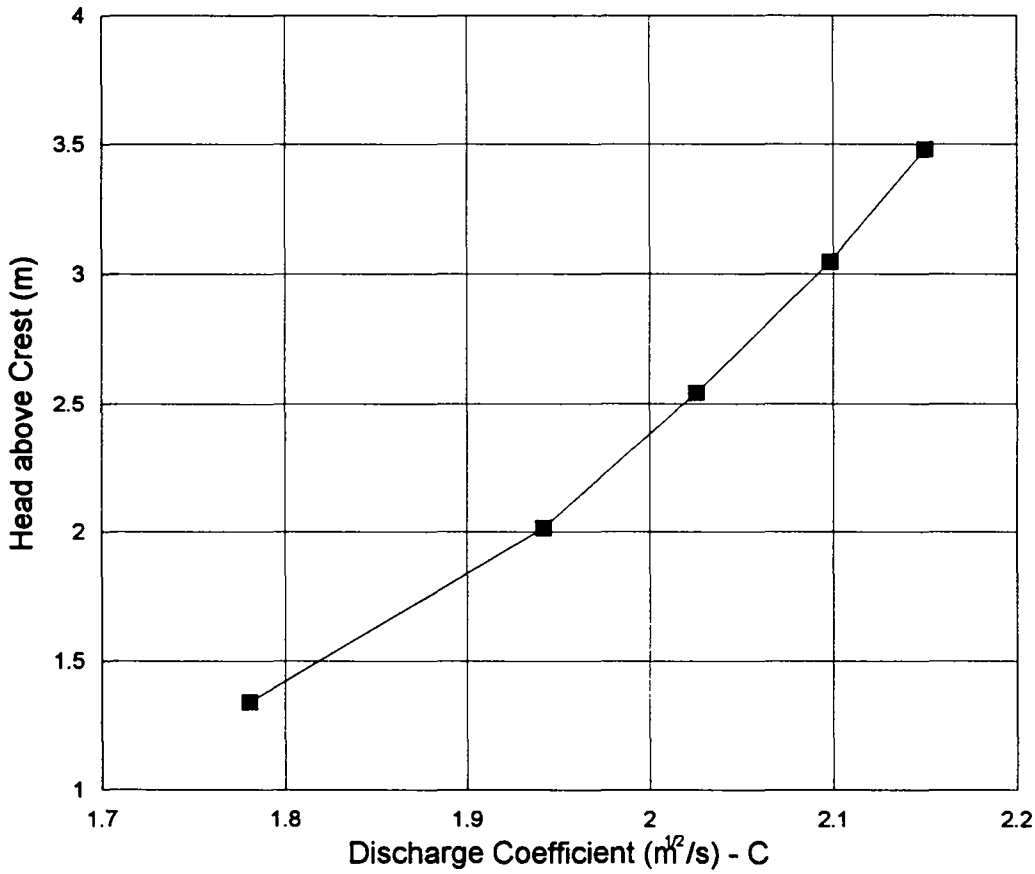


Figure 33. Head versus discharge coefficient (C) for the spillway crest for the equation $Q = CLH^{3/2}$.

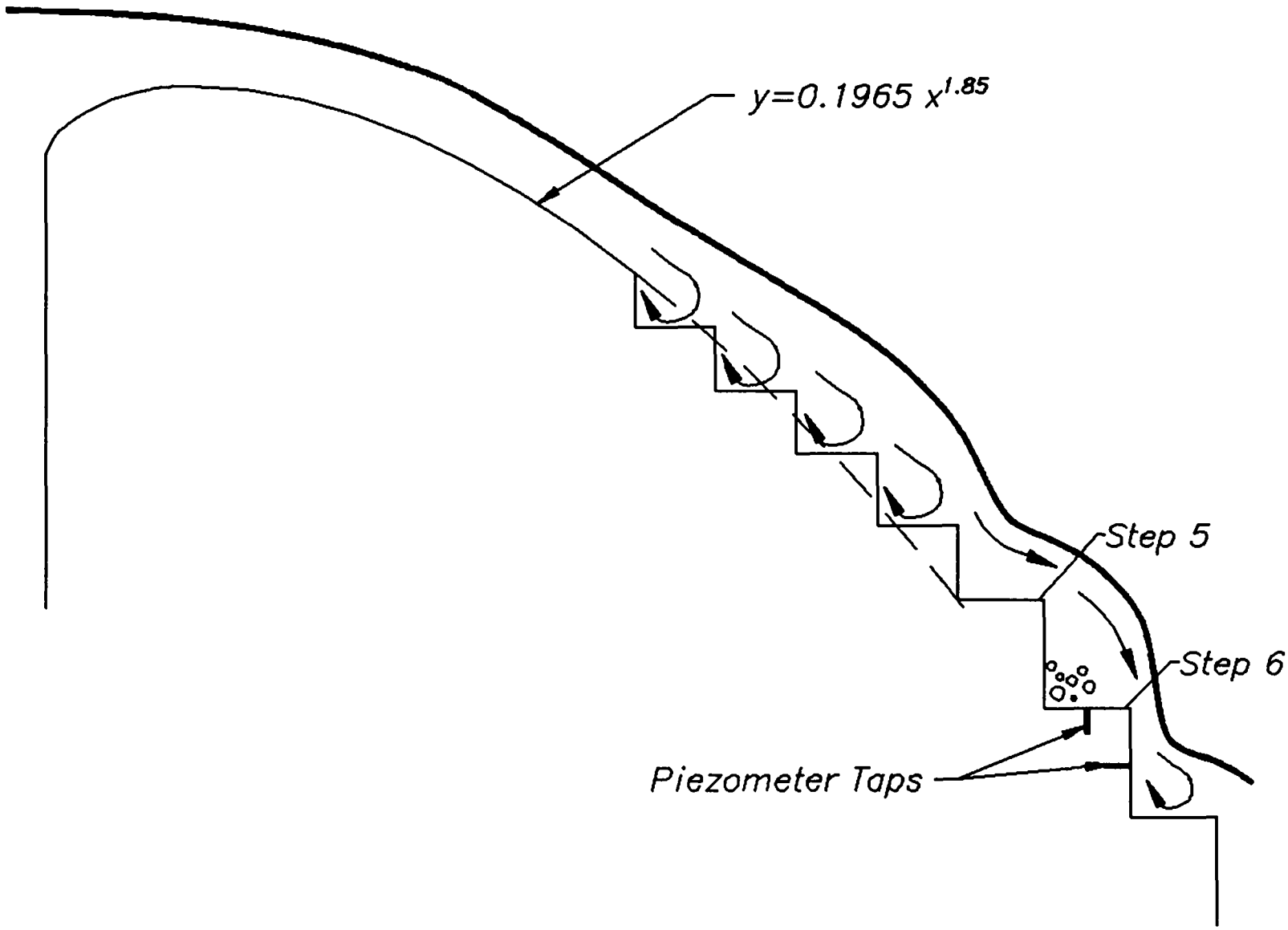


Figure 34. Flow pattern over crest and transition to spillway slope (water surface is not to scale).

(a)

(b)

Figure 35. A diversion flow of 235 m^3/s showing a) the upstream cofferdam and diversion channel, and b) outflow from the twin culverts into the downstream diversion channel.

PEER REVIEW DOCUMENTATION

PROJECT AND DOCUMENT INFORMATION

Project Name Pilar Dam WOID Pilar

Document Hydraulic Model Study of Pilar Dam

Document Date January 1996 Date Transmitted to Client _____

Team Leader Leslie Hanna\Cliff Pugh Leadership Team Member _____
(Peer Reviewer of Peer Review/QA Plan)

Peer Reviewer Phil Burgi Document Author(s)/Preparer(s) Leslie Hanna\Cliff Pugh

Peer Reviewer _____

Peer Reviewer _____

REVIEW REQUIREMENT

Part A: Document Does Not Require Peer Review

Explain _____

Part B: Document requires Peer Review : SCOPE OF PEER REVIEW

Peer Review restricted to the following Items/Section(s): Reviewer:

REVIEW CERTIFICATION

Peer Reviewer - I have reviewed the assigned Items/Section(s) noted for the above document and believe them to be in accordance with the project requirements, standards of the profession, and Reclamation policy.

Reviewer: _____ Review Date: 1/6/97
Signature

Preparer: I have discussed the above document and review requirements with the Peer Reviewer and believe that this review is completed, and that the document will meet the requirements of the project.

Team Member: _____ Date: 1/8/97
Signature

Attachment 1

July 1996 Meeting Notes Summarizing Modifications
and Test Plan for Pilar Dam Hydraulic Model Study

Pilar Dam Model Meeting Notes

Date: 7/17/96

Participants: Clifford A. Pugh, Bureau of Reclamation
Leslie J. Hanna, Bureau of Reclamation
Aluisio Pardo Canholi, THEMAG

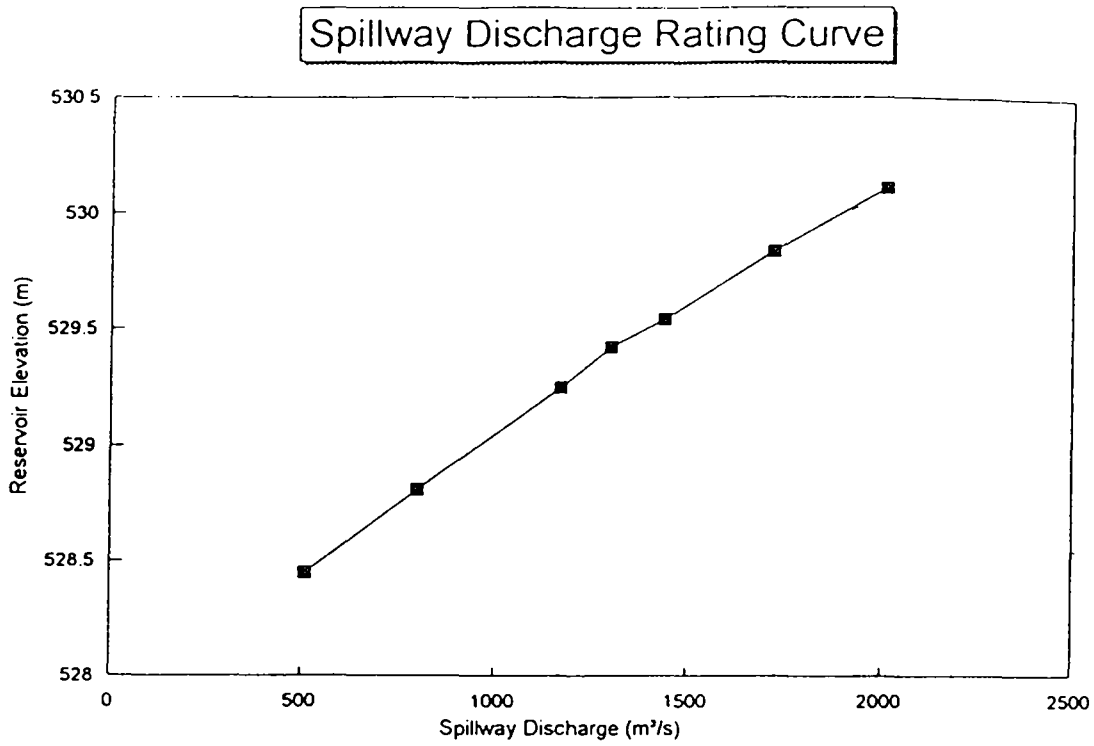
Main subjects:

1. THEMAG visited the Bureau of Reclamation during the period July 10-17, 1996.
2. The model construction is finished and the calibration tests have been completed.
3. Some initial results of the calibration phase are attached. The crest discharge capacity was close to the anticipated discharge. At water level (W.L.) 530.00 m, the flow rate is about 2,000 m^3/s (enclosure #1).
4. Preliminary tests have been completed on the diversion structures. The design capacity was confirmed with a little additional head required. To pass Q=235 m^3/s, the upstream W.L.=470.50 m. This implies that the free board is 1.0 m if the cofferdam crest is at elevation 471.50 m (enclosure #2).
5. The model was observed operating for the original design at discharges ranging from 1,000 to 2,500 m^3/s. At the maximum discharge of 2,500 m^3/s, the dam is not overtopped.
6. Photographs and videos of the model were taken at each flow condition.
7. In general, the model performance upstream and over the spillway is satisfactory. The flows downstream are safe, but some aspects can be improved, as the following paragraphs discuss. The recorded velocities downstream are presented in enclosure #3 (original design).
8. <u>Energy dissipation at the toe of the spillway</u>
    - The main objective of this analysis was to evaluate the necessity for a slab at the toe.
    - The initial observation led to the conclusion that if the hydraulic cushion is improved, the slab is probably not necessary.
    - To improve the cushion, the model was modified by adding an excavated trench at the toe of the dam (modification #1). The general dimensions of this modification are 15.0 m wide by approximately 5.0 m deep, with a bottom elevation of 465.00 m and slope 1:1 to the natural topography.
    - The preliminary tests made according to the above modification show an excellent performance. The observed discharges were 1,000 cms and 2,000 m^3/s.
    - Videos and photos of modification #1 were taken.
9. <u>Flow over the spillway</u>
    - Energy dissipation and flow down the spillway looks good with the exception of the impact on the abutments.
    - Considering the presence of loose material on the left abutment and the

impact in this area, it was decided to move the spillway location by 10.0 m to the right and to converge the training walls, keeping the same spillway crest length. Both walls were converged about 16 degrees. The spillway width was narrowed from 180 m wide at the crest to 140 m wide at elevation 471.5 m (modification #2).

- At the toe of the spillway, all of the flow will enter the tailwater without striking the abutments. The concentrated flow at the converging training walls will be directed to the deepest parts of tailwater at each side of the spillway. At the right side, into the diversion channel, and at the left side, into the river channel.
- Videos and photos of the tests with modification #2 were taken.

10. Future Activities
    - The table presented in enclosure #4 has a schematic test program that will be carried out with the model after "tuning" modification #2.
    - The scope and the goals of each investigation were discussed.
    - The final report containing all of the results will be edited by October 1.
    - Preliminary test results will be faxed to THEMAG as they are completed.



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Enclosure #1

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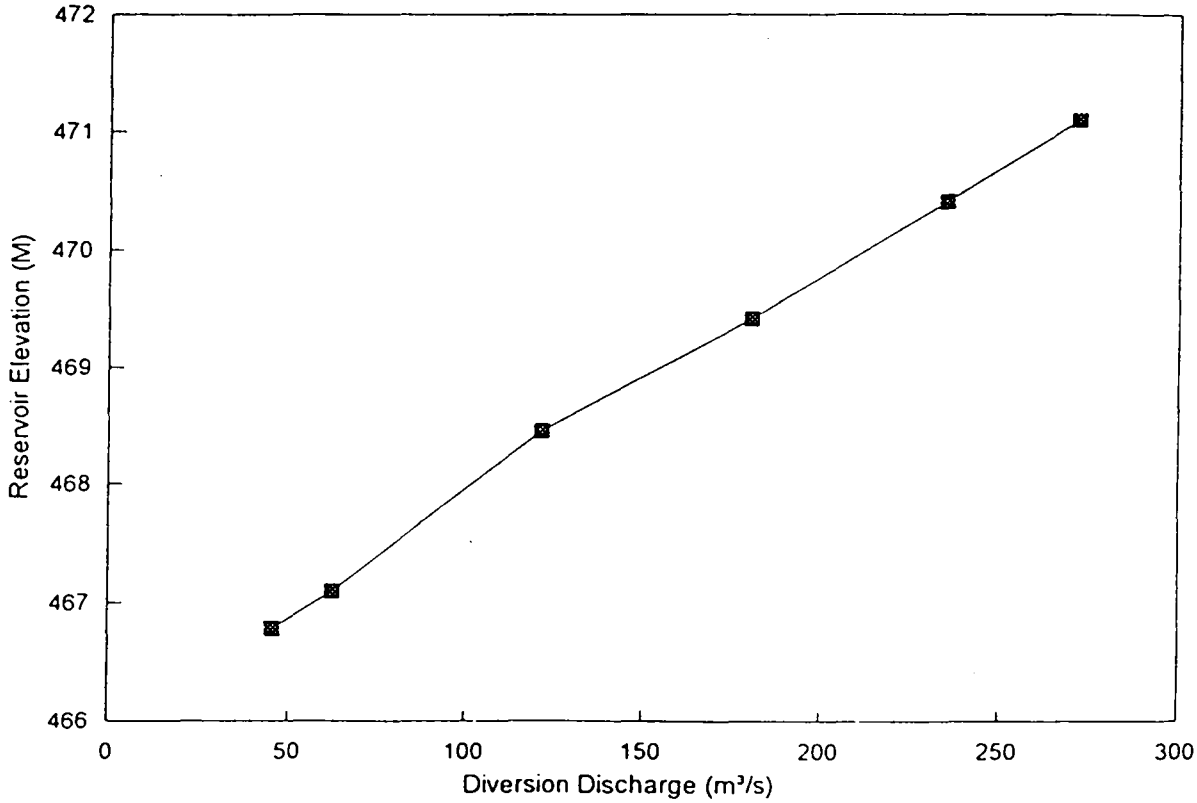
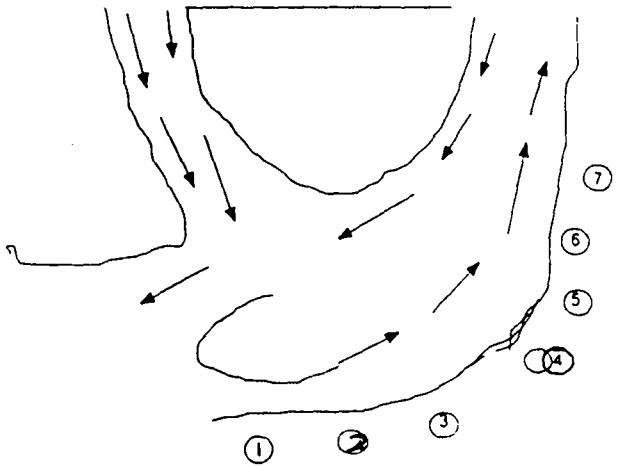
Diversion Discharge Rating Curve



Enclosure #2

Velocities measured next to bank opposite downstream diversion channel

Discharge (M^3/s)	Position	ORIGINAL DESIGN Model Velocity(M/s)	ORIGINAL DESIGN Prototype Velocity (M/s)	MODIFICATIONS I Model Velocity(M/s)	MODIFICATIONS I Prototype Velocity (M/s)
1000	1	0 07	0 4427188724	0 09	0.5692099788
	2	0 11	0 6957010852	0 16	1 0119288513
	3	0.22	1 3914021705	0 16	1 0119288513
	4	0 25	1 5811388301	0 19	1.2016655109
	5	0 24	1 5178932769	0.15	0.9486832981
	6	0 3	1 8973665961	0.18	1.1384199577
	7	0 34	2 1503488089	0 19	1 2016655109
2000	1	0 19	1 2016655109	0 1	0.632455532
	2	0 2	1 2649110641	0 13	0 8221921916
	3	0 16	1 0119288513	0 11	0.6957010852
	4	0 18	1 1384199577	0 18	1.1384199577
	5	0 31	1 9606121493	0 19	1 2016655109
	6	0 2	1 2649110641	0 17	1 0751744045
	7	0 25	1 5811388301	0 18	1 1384199577



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Enclosure #3

Model Tests General Program

Rate Discharges: 200, 500, 1000, 2000, 2500 cms

Flow Over Steps	Flow Downstream	Diversion Flows
Subjects	**Subjects**	**Subjects**
• Approach flows • Pressures on the steps (mainly steps transition) • Water surface profiles (at training walls and away from)	• Recording velocity measurements over a net (keep all test measurements at the same points) • Fluctuations pressures at the toe • Qualitative test with loose material at (d_{50}=1.0 m/e=2.0 m) the pool (*)	• Velocities close to the cofferdam Q=235 cms • Rate discharge culvert curve • Qualitative test with Q=800 cms

e=2.0 m
d_{50}=1.0 m

Enclosure #4