



Technical Report CHL-99-7  
April 1999

**US Army Corps  
of Engineers**  
Waterways Experiment  
Station

# **Dam Safety Assurance Model Study of Tygart Dam, Tygart River, Grafton, West Virginia**

*by Herman O. Turner, Jr.*

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Prepared for U.S. Army Engineer District, Pittsburgh

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by Herman O. Turner, Jr.

U.S. Army Corps of Engineers  
Waterways Experiment Station  
3909 Halls Ferry Road  
Vicksburg, MS 39180-6199

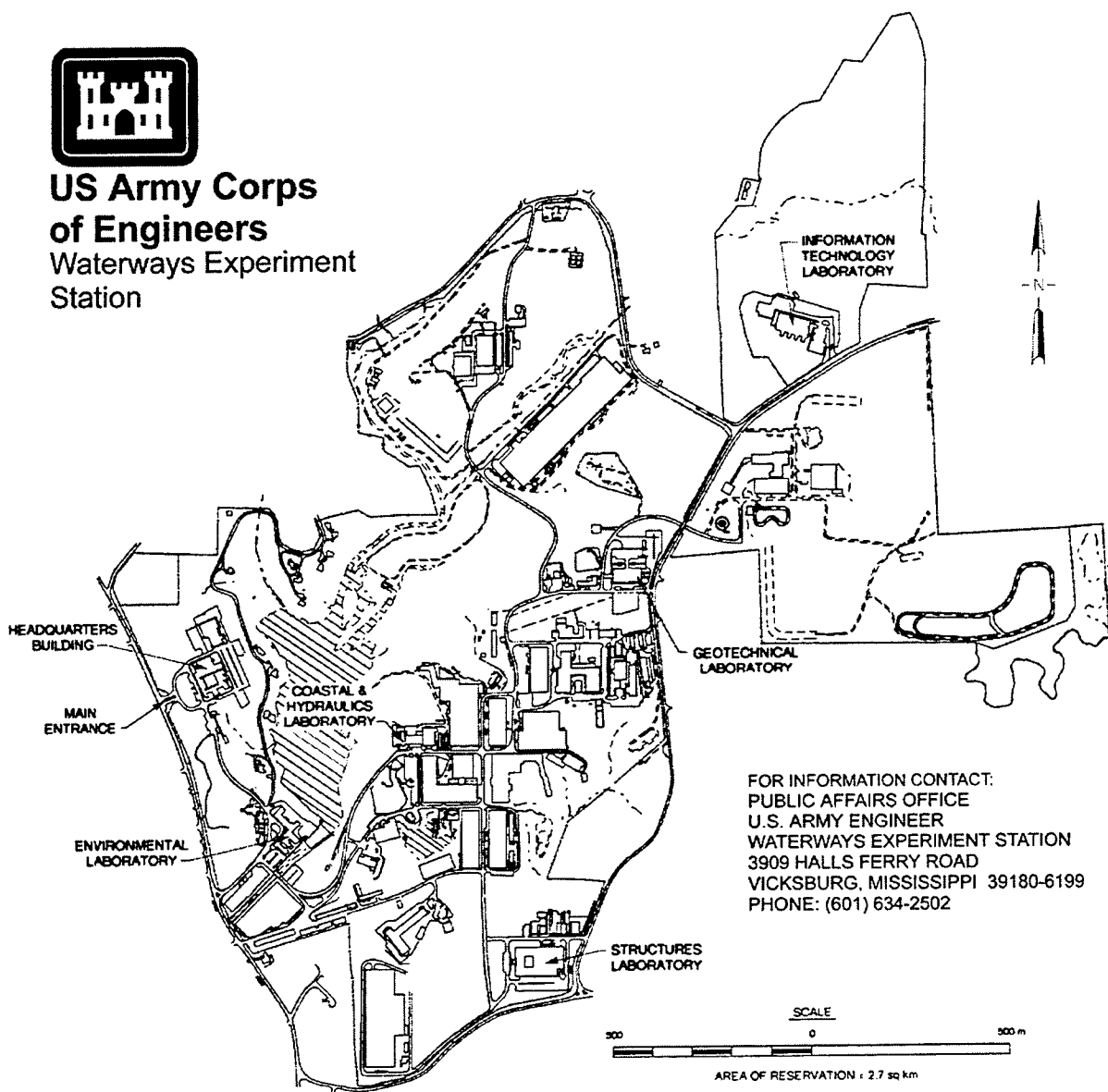
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# Preface

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The model investigation reported herein was authorized by Headquarters, U.S. Army Corps of Engineers, 29 May 1996, at the request of the U.S. Army Engineer District, Pittsburgh, through the U.S. Army Engineer Division, Ohio River.

The model investigation was conducted during the period June 1996 to August 1997 in the Coastal and Hydraulics Laboratory (CHL) of the U.S. Army Engineer Waterways Experiment Station (WES) under the general supervision of Dr. James R. Houston, Director, CHL, and Messrs. Richard A. Sager (retired) and Charles C. Calhoun, Jr. (retired), Assistant Directors, CHL; Dr. Phil G. Combs, Chief, Hydraulics Structures Division (HSD), CHL; and under the direct supervision of Messrs. Bobby P. Fletcher (retired), Chief, Spillway and Channels Branch (SCB), HSD, and D. D. Davidson (retired), Acting Chief, SCB. The tests were conducted by Messrs. Herman O. Turner, Jr., and Kevin L. Pigg, both of SCB. Dr. Frank M. Neilson (deceased) calculated the total discharge. This report was prepared by Mr. Turner and reviewed by Dr. John E. Hite.

Messrs. W. Leput, M. Zaitsoff, R. Povirk, R. Rush, J. Kosky, A. Krysa, D. Carlson, W. McCann, and R. Burstynowicz, Pittsburgh District, visited WES during the course of the model study to observe model operation and correlate results with design studies.

At the time of publication of this report, COL Robin R. Cababa, EN, was Acting Director of WES.

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# 1 Introduction

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## Prototype

Tygart Dam (Figure 1) is located on the Tygart River approximately 3.6 km (2.25 miles) south of Grafton, WV (Figure 2). Construction on Tygart Dam started in 1935 and was completed in February 1938. It is a concrete gravity dam 586 m (1,921 ft) in length and forms Tygart Lake. At the spillway crest elevation (el) of 1167 ft,<sup>1</sup> the lake area is 1,388 ha (3,430 acres). The lake and dam were constructed as a multipurpose lake project for flood control, navigation, water supply, and recreation.

Water is periodically released from Tygart Lake through the dam by eight rectangular conduits, 1.73 m wide by 3.05 m high (5.67 by 10.0 ft). Flow in the conduits is controlled by slide gates. Details of the dam and conduits are shown in Plates 1-3. Releases from the conduits are initially deflected into the stilling pool by parabolic deflectors (Figure 3) placed at the conduit exits. The parabolic deflectors spread the conduit flows and dissipate the conduit energy. A stilling weir located 122 m (400 ft) downstream from the axis of the dam has a crest of el 986. The stilling weir forms a stilling pool to dissipate the energy of the conduit and spillway flows. The water exits the stilling pool over the stilling weir and into the Tygart River. The streambed has an el of 960 (293 m) downstream from the structure.

The Tygart spillway is an uncontrolled ogee type. The spillway crest (Plates 2 and 3) is at el 1167 and is 136.7 m (488.33 ft) wide (Plate 1). Spillway abutments and parapet walls at each side of the spillway are at el 1194.25 (Plate 2). Training walls are located on each side of the overflow section. The toe of the overflow section transitions into nine dentates formed to concentrate flow away from the conduit openings and parabolic deflectors, as shown in Plate 2. The spillway was designed for a maximum flow of 6,089 cu m/sec (215,000 cfs) at pool el of 1190.0.

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<sup>1</sup> All elevations (el) cited herein are in feet referenced to the National Geodetic Vertical Datum (to convert feet to meters, multiply number of feet by 0.3048).

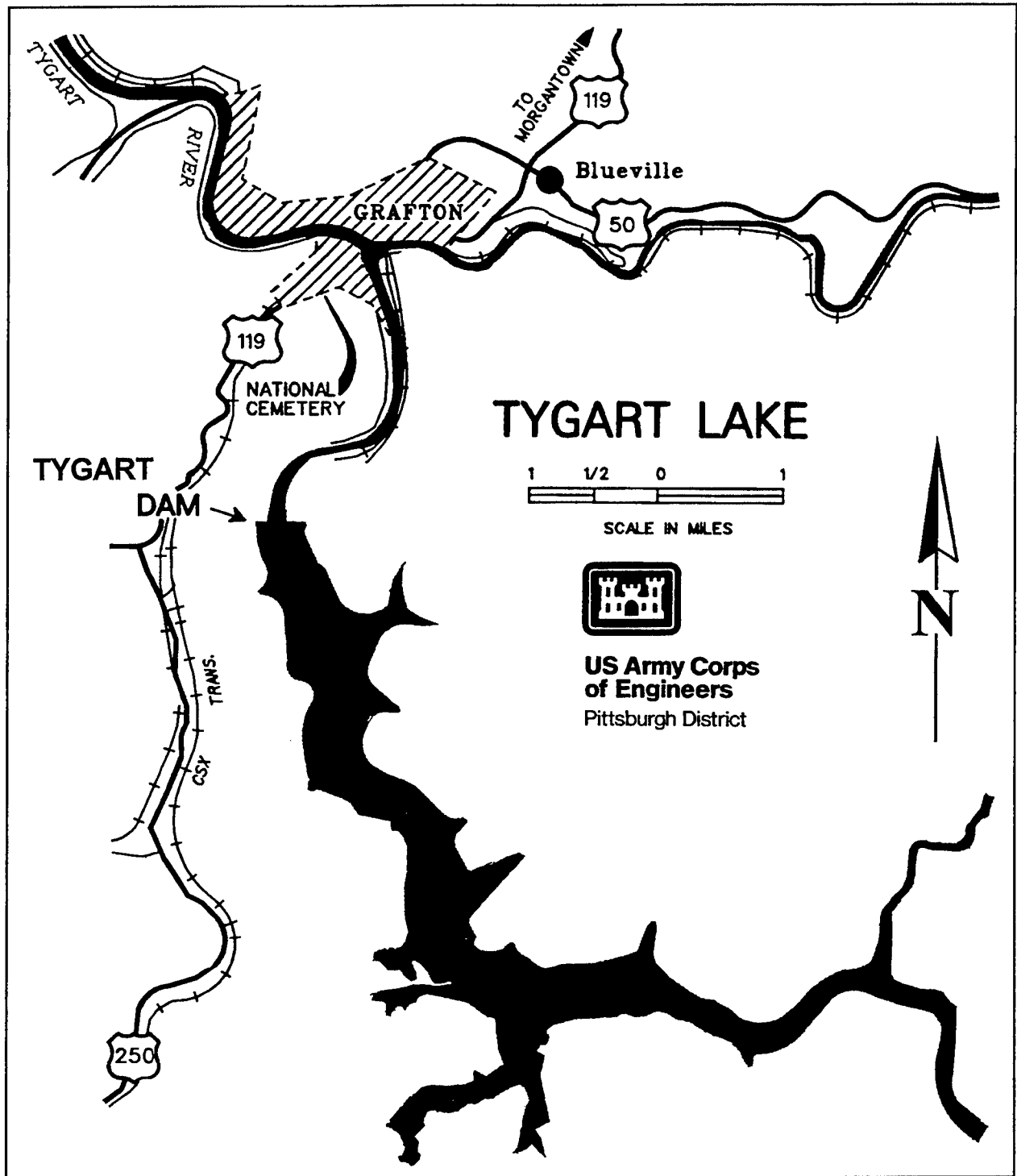


Figure 1. Project map (to convert miles to kilometers, multiply by 1.609)

In 1956, the National Weather Service (NWS) published generalized estimates of probable maximum precipitation (PMP) for areas of the United States east of the 105th meridian in Hydrometeorological Report (HMR) No. 33. Later, at the request of the U.S. Army Corps of Engineers, the NWS published

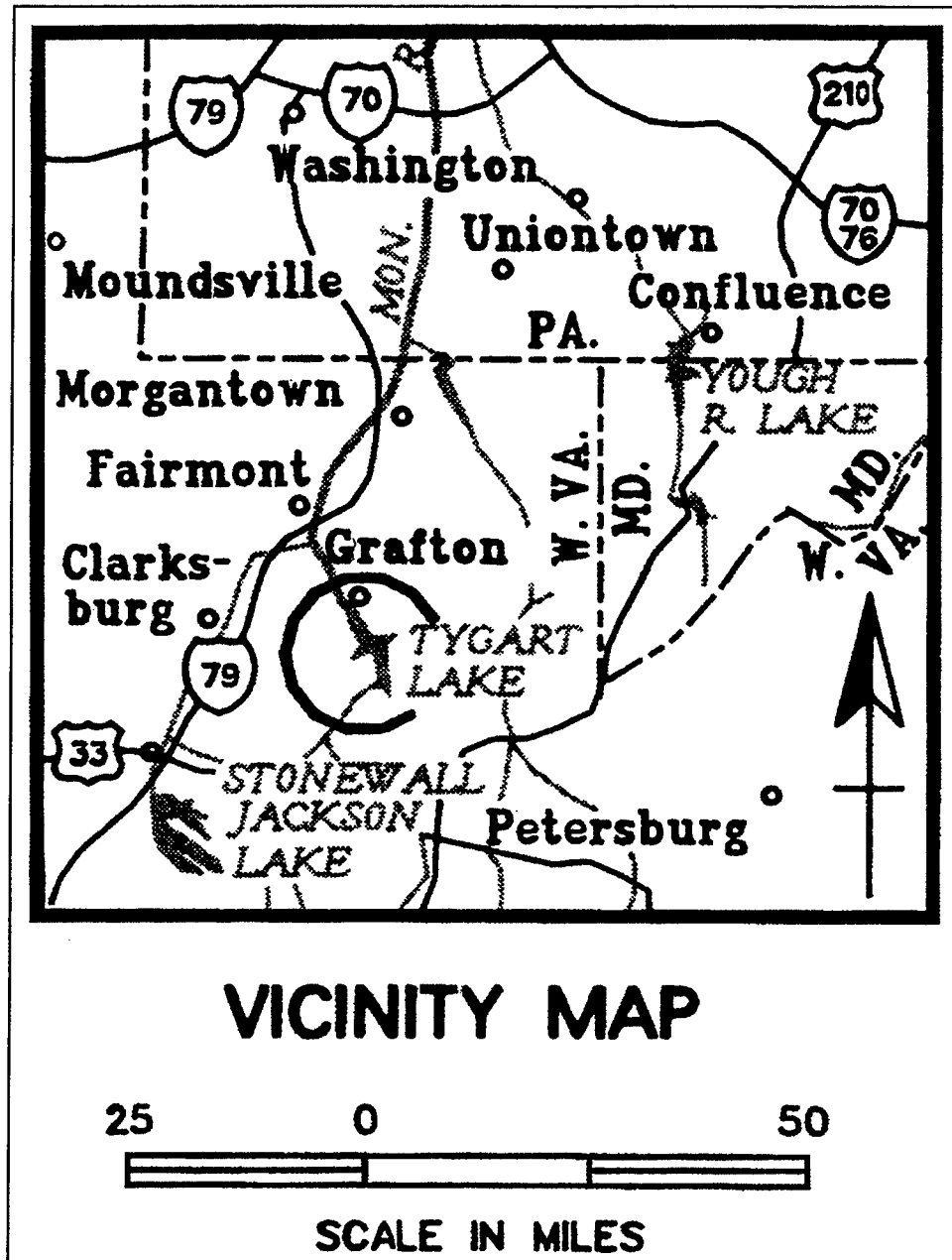


Figure 2. Vicinity map (to convert miles to kilometers, multiply by 1.609)

HMR No. 51, dated June 1978, which revised and expanded PMP estimates. The dam safety assurance analysis used HMR No. 51 to derive the probable maximum flood (PMF) and subsequent hydrologic deficiencies of Tygart Dam.

Because of hydrologic deficiencies, Tygart Dam will not safely pass the PMF of 13,175 cu m/sec (373,120 cfs) as revised by the Dam Safety Assurance Study. The PMF will result in overtopping the dam for 11 hr and reach a maximum water surface approximately el 1196.95 or 0.82 m (2.7 ft) over the dam. It is

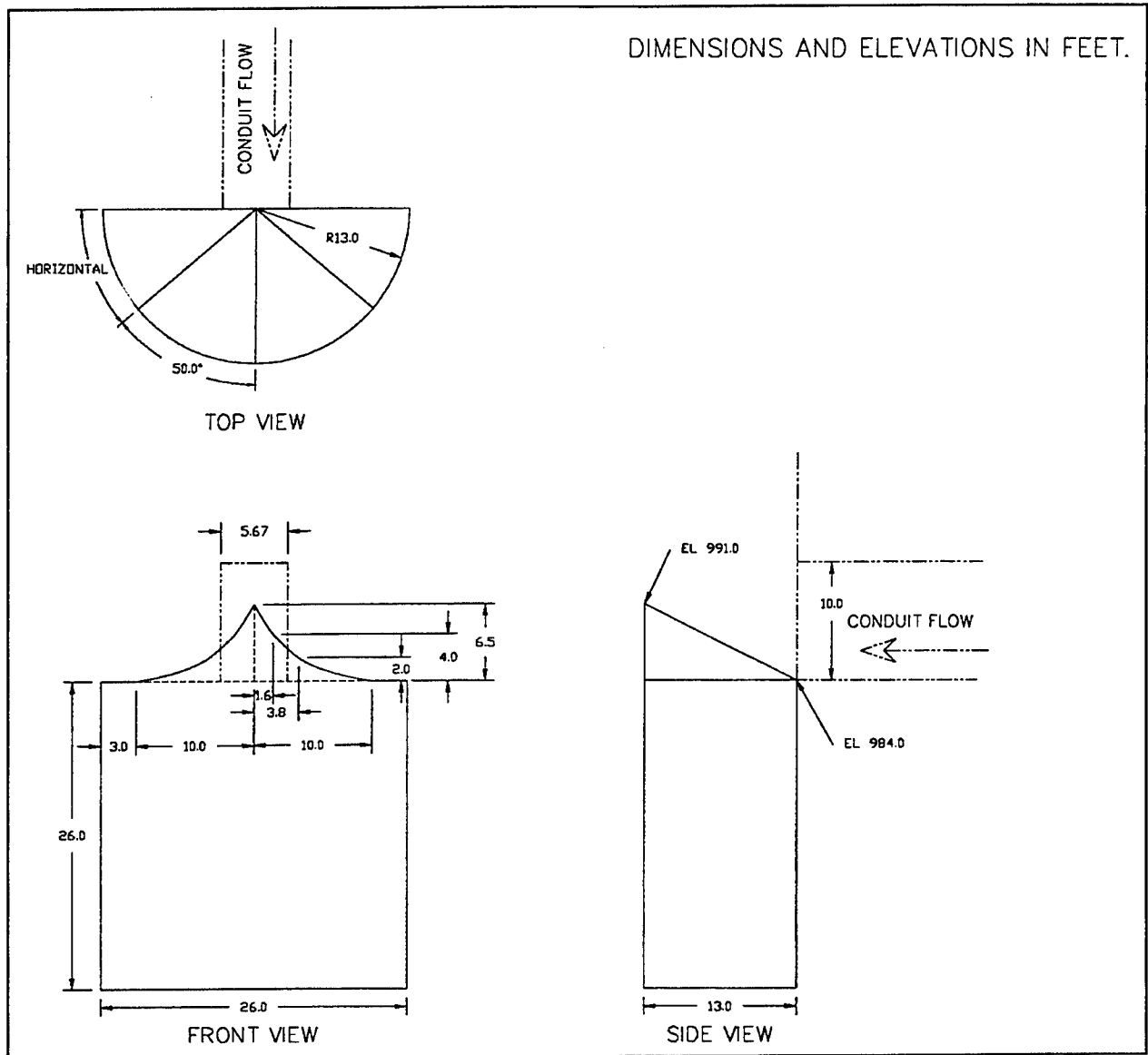


Figure 3. Parabolic deflector

anticipated during the PMF, that the overtopping flows will scour foundation material from the toe of the dam. This is a critical area for stability of the dam. If the stability is lost, failure of the dam is expected to occur because of the sliding or overturning of one or more monoliths. Based on other documented dam failures by the Bureau of Reclamation, the failure of Tygart Dam would be sudden with little advance warning.

The U.S. Army Engineer District, Pittsburgh, evaluated several alternatives to ensure that Tygart Dam meets the revised safety standards. These alternatives included no action, raising the dam, spillway modification, auxiliary spillway, overtopping, and replace the dam.

If no action is taken, the structure will continue to operate until overtopping occurs. As originally designed, the existing spillway can safely pass flows up to 7,642 cu m/sec (269,850 cfs) which is 72 percent of the PMF. At this discharge, the corresponding pool is at the elevation as the top of dam roadway (el 1090). Rainwater drains along both sides of the roadway would allow the rising pool to seep onto the roadway and exit by way of the downstream drains. For flows greater than 9,047 cu m/sec (319,440 cfs), overtopping occurs that will result in the potential failure of the dam.

Raising the dam would allow the existing spillway to pass the PMF without overtopping the dam. With this alternative, the top of the dam would be raised 1.84 m (6.05 ft) to el 1200.3 to provide for the raised pool.

The spillway modification alternative involves lowering the spillway crest, adding a pier at the center of the crest. Two inflatable rubber dams would be installed on the lowered crest and would provide storage up to the level of the existing crest. When overtopping occurs, the inflated dams would be deflated to pass the PMF.

With the auxiliary spillway alternative, another spillway adjacent to the existing dam would be constructed. The combined existing and auxiliary spillways would safely pass the PMF.

With the overtopping alternative plan, the downstream side of the dam would be protected from scour and erosion. Protection along the downstream toe would be provided for the full width of the dam. Design features of this plan will resist the impact of the overtopping flow and carry the water to the stilling basin or downstream channel.

## **Purpose and Scope of Model Investigation**

The purpose of this model investigation was to study alternative ways of conveying the revised PMF when the existing dam is overtopped.

The main objectives of the model study were as follows:

- a.* Determine necessary modifications to the dam and spillway to safely pass the PMF.
- b.* Determine modifications needed to the stilling basin and exit channel for the increased flows with the new PMF.

The model investigation involved identifying the hydraulic problems associated with using an existing structure required to convey 74 percent more than for which the spillway was designed.

## 2 Model Description

---

The investigation was conducted using a 1:60 scale physical model (Figures 4 and 5). The basis for choosing this size model will be discussed in a following section. The model reproduced approximately a 610-m (2,000-ft) length of approach channel, the dam (Figure 4), and 610 m (2,000 ft) of exit channel (Figure 5). The model topography was modeled to existing topography supplied by the Pittsburgh District. Sheet metal templates were prepared according to location and elevation data, installed, and paved with a thin layer of cement grout (Photo 1). Sheet metal was also used to fabricate the model dam and endsill. The stilling basin was constructed of plastic-coated marine plywood. The spillway dentates were molded with plastic resin. The roadways, parapet walls, and training walls were constructed of clear plastic. All model components were constructed to close tolerances. Very smooth materials were utilized to minimize geometric distortion.

The existing design, Type 1, represents the current prototype conditions. The existing topography was reproduced according to a survey conducted by the Pittsburgh District. The topography was only reproduced up to sufficient elevations to contain the expected water levels and provide reasonable freeboard. This modification had no impact on the model results.

A closeup view of the upstream face of the spillway and the eight rectangular and two circular conduit openings is shown in Photo 2. At each side of the spillway are circular conduits designed for low releases. Currently, the circular conduits are not operational in the prototype. The downstream view of the model (Figure 5) shows the downstream channel alignment. In Photo 3, the details of the spillway and stilling basin such as the dentates, deflectors, and stilling weir are shown.

### Model Appurtenances

Water used in the operation of the model was supplied by a recirculating system. Flow rate in the model was measured with venturi flowmeters. Steel rails graded to specific elevations were placed along both sides of the model to serve as supports for measuring devices and to provide a convenient means of establishing stations and elevations in the model. Velocities were measured with

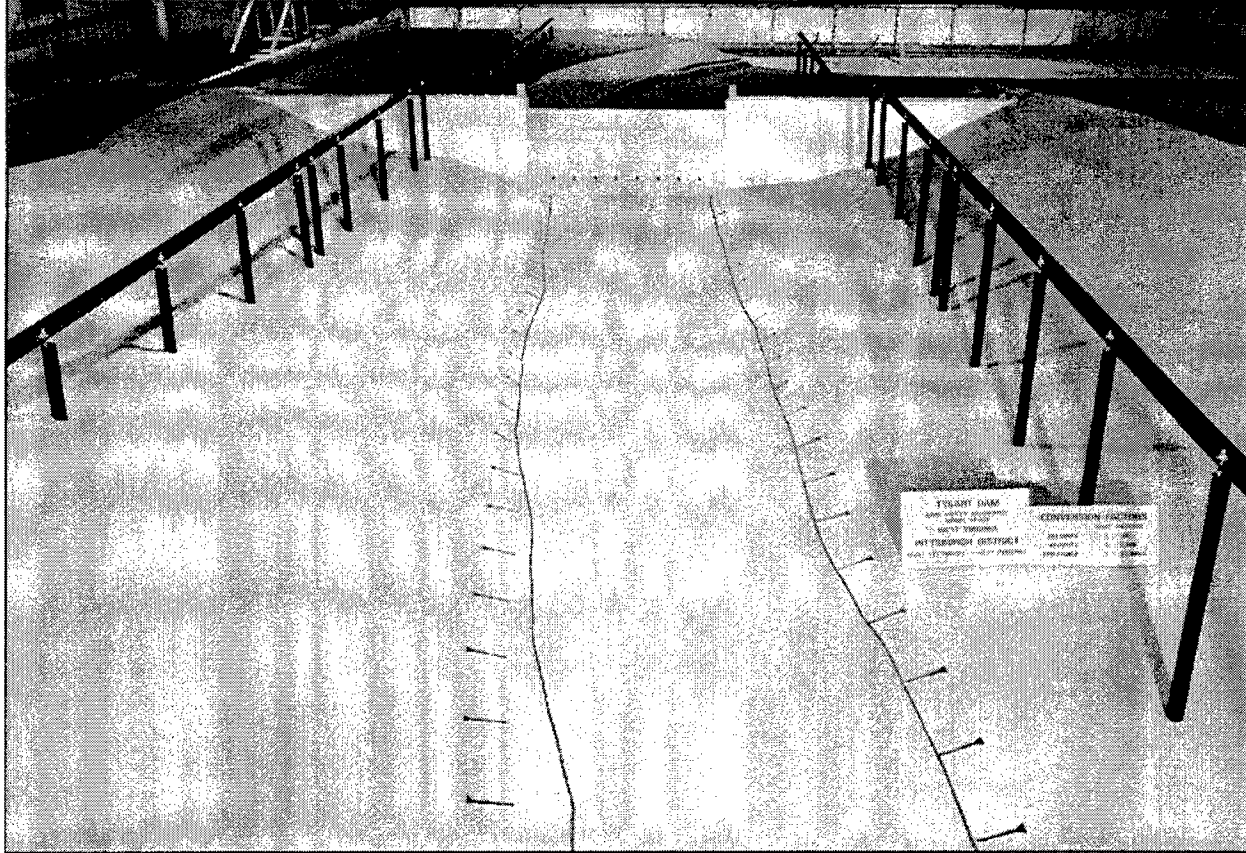


Figure 4. Type 1 existing conditions, dry bed, upstream view

an electronic velocity meter and pitot tube. Tailwater elevations were regulated by an adjustable gate at the end of the flume. Water-surface elevations were measured with point gauges and electronic water-level detectors. Various modifications along with different flow conditions were recorded by sketches and photographs.

## Scale Relations

The model scale and size are designed to investigate stilling basin performance and flow patterns and water velocities in the exit channel. These flow characteristics are dominated by inertial and gravitational forces. Froude-scale modeling is the most appropriate technique for quantifying these types of flow characteristics. The Froude number

$$F_n = \frac{V}{\sqrt{g L}} \quad (1)$$

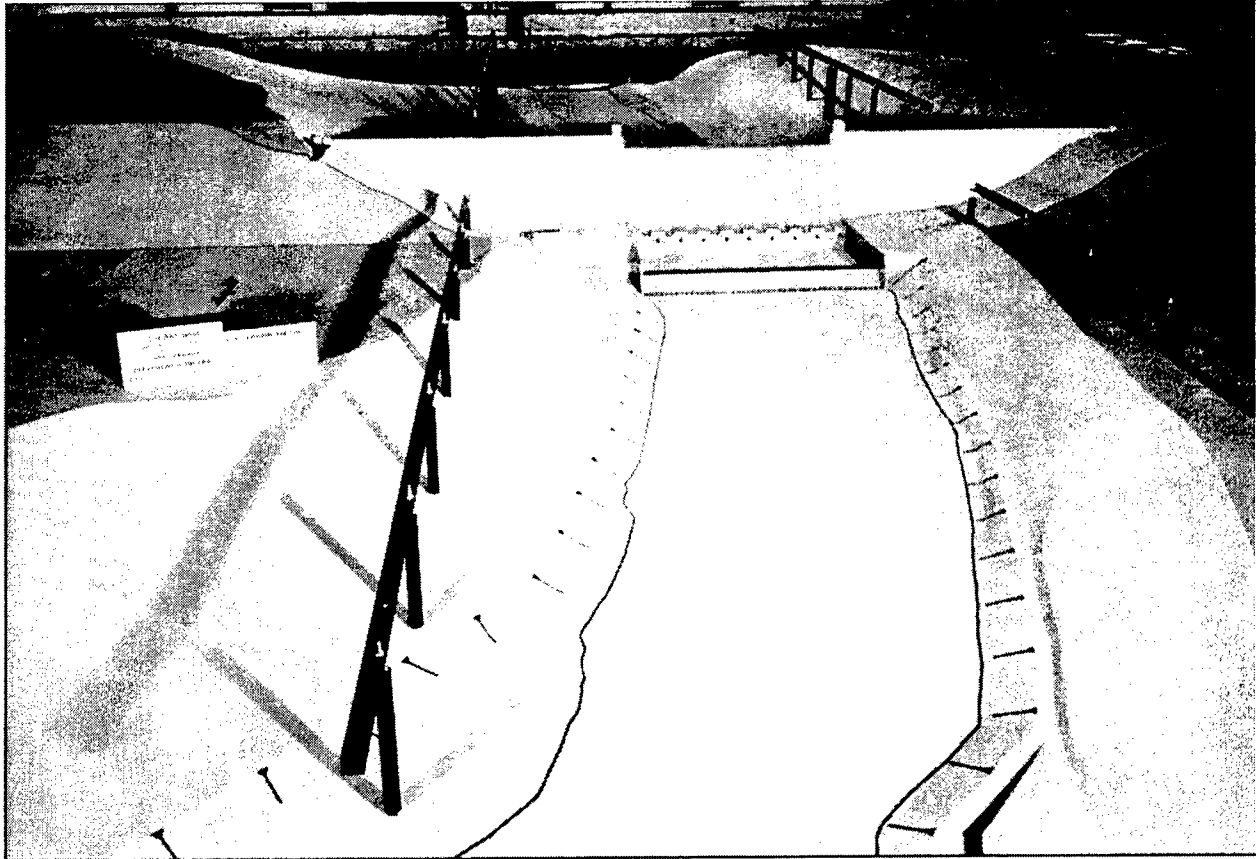


Figure 5. Type 1 existing conditions, dry bed, downstream view

where

$F_n$  = Froude number

$V$  = water velocity

$g$  = acceleration because of gravity

$L$  = characteristic length within model

expresses the ratio of inertial to gravitational forces. When the Froude number in the model is equal to that of the prototype, the flow patterns in the model will be similar to those in the prototype.

A properly designed Froude-scale model will achieve geometric and kinematic similitude. Dynamic similitude cannot be practically achieved in this type of model investigation. The model must therefore be designed so that the effects of not achieving total dynamic similitude are minimal. This is accomplished by ensuring that the flow conditions in the model are in a wholly rough turbulent regime and the viscous effects do not affect flow patterns and velocities.

The Reynolds number

$$R_n = \frac{V \times D}{\nu} \quad (2)$$

where

$R_n$  = Reynolds number

$V$  = water velocity

$D$  = depth

$\nu$  = kinematic viscosity

expresses the ratio of viscosity to inertia. Keeping the Reynolds number in the turbulent zone (>2,000) for all flow conditions was desirable to reduce viscous scale effects. For this study, a scale ratio of 1:60 was selected to satisfy both Froude criteria and minimize viscous effects while providing a practical model size and cost. The resulting model simulates 610 m (2000 ft) of upstream topography, 610 m (2,000 ft) of downstream channel, and flow rates of up to 420,000 cfs.

The equations of hydraulic similitude, based on Froudian relations, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for transferring model data to prototype equivalents are as follows:

Characteristic	Dimension <sup>1</sup>	Scale Relations Model:Prototype
Length	$L_r$	1:60
Area	$A_r = L_r^2$	1:3,600
Velocity	$V_r = L_r^{1/2}$	1:7.746
Discharge	$Q_r = L_r^{5/2}$	1:27,885.5
Time	$T_r = L_r^{1/2}$	1:7.746

<sup>1</sup> Dimensions are in terms of length.

### 3 Experiments and Results

---

Initial experiments were conducted to observe and verify existing spillway and sluice calibrations for Tygart Dam. Total discharge for the structure were determined according to the following formula:

$$Q_{total} = Q_{conduits} + Q_{spillway} + Q_{overflow} \quad (3)$$

where

$$Q_{conduit} = C_0 (Z_c)^{1/2} = 2700 = (Z-989)^{1/2}$$

$$C_0 = \text{Conduit Discharge Coefficient} = C \times \text{Area} = 2700$$

$$Z = \text{WS elev} - \text{Conduit Outlet Elevation} = \text{WS elev} - 989$$

$$Q_{spillway} = C_1 (W - K_a K_c) H_e^{3/2} = C_1 (489.3 - 0.2 (Z-1167)) (Z-1167)^{3/2}$$

$$C_1 = \text{Spillway Discharge Coefficient}$$

$$W = \text{Crest Length} = 489.3 \text{ ft}$$

$$K_a = \text{Abutment Contraction Coefficient} = 0.2$$

$$H_e = \text{Energy Head Crest} = \text{WS elev} - \text{Crest Elev}$$

$$Q_{overflow} = C_2 (L - K_R H_c) (H_c)^{3/2}$$

$$= 2.8 (1280 - 0.2 (\text{WS} - 1194.25)) (\text{WS} - 1194.25)^{3/2}$$

$$C_2 = \text{Parapet Wall Flow Coefficient} = 2.80$$

$$L = \text{Parapet Wall Length} = 1280 \text{ ft}$$

$$K_R = \text{Parapet Wall Contraction Coefficient} = 0.2$$

$$H_c = \text{Water Surface Elev} - \text{Parapet Wall Elevation}$$

The model rating curve was developed by setting the model discharge using verturi meters. After the pool settled, the pool elevation was recorded using adjustable point gauges calibrated to a known elevation. This procedure was continued throughout the range of pool elevations. The corresponding prototype discharges and pool elevations were calculated using the above model:prototype scale relationships. These data were plotted with the calculated values and are shown in Plate 4. The correlation between calculated and observed values indicates that the selected flow coefficients were accurate.

## Flow Conditions with Type 1 Design

The performance of the Type 1 (existing) design was documented by observing flow over the range of discharges shown in Table 1. The tailwater elevations were set according to a rating curve furnished by the Pittsburgh District (Plate 5).

Condition	Total Flow, cfs	Reservoir Pool	Tailwater Elev	Comments
A	41,580	1167.0	987.0	Sluice flow only
B	250,780	1190.0	1020.0	Stilling basin walls overtopped
C	319,440	1194.25	1026.5	Dam overtopped
D	373,120	1196.95	1031.5	PMF flow

Condition A flow is shown in Photos 4-5 and represents flow from the rectangular conduits with the upstream pool at the crest el of 1167. These photos show the spreading of the conduit jet caused by the deflectors. The flow in the downstream channel has a tailwater el of 987, and this discharge is confined within the channel banks.

Condition B flows are shown in Photos 6-7. The discharge for the Condition B flow was determined by gradually increasing the discharge from the Condition A flow until the stilling basin walls overtopped. As the discharge was increased, the tailwater was adjusted according to Plate 5. This flow was determined to be 7,102 cu m/sec (250,780 cfs) with a corresponding pool el of 1190. The tailwater in the downstream channel with this discharge is at el 1020. Condition B causes flooding near the channel (see Grafton Water Treatment Plant, Photo 7). With flow Condition B, the weir at the end of the stilling basin causes a forced hydraulic jump in the basin. A secondary jump also occurs in the channel downstream as the flow over the weir enters the tailwater.

Photos 8-9 show conditions just before the pool elevation overtops the dam (Condition C). The discharge for Condition C was determined by gradually increasing the discharge until the pool elevation was the same as the top of the

parapet walls. The tailwater elevation was adjusted according to Plate 5. The pool elevation is 1194.25, and the flow rate is 9,047 cu m/sec (319,440 cfs). Action in the stilling basin is stronger, and the secondary jump has moved farther downstream because of the higher flow.

The PMF flow conditions, Condition D, are shown in Photos 10-11. At Condition D, the flow rate is 10,567 cu m/sec (373,120 cfs), the pool elevation is 1196.95, and the tailwater is at el 1031.5. The overtopping flow comes over the parapet walls and down the downstream face of the dam. Erosion of the natural terrain is expected to take place as the water impacts the ground surface and attempts to reenter the downstream channel. Some of the overtopping flow flanks the parapet wall on the right side and moves through the roadway opening. As shown in Photo 12, the flow is conveyed along the access roadway before spilling back down the terrain. On the left descending side of the dam, no flanking flow occurs because of higher terrain and the absence of an access roadway. The downstream flow conditions show that a forced hydraulic jump is still occurring in the stilling basin, the secondary jump has moved even farther downstream, and significant surface turbulence and wave action are associated with the secondary jump.

Velocities in the downstream channel were obtained to document the Type 1 design. These point velocities were recorded 0.6 m (2 ft) from the surface (T) and 0.6 m (2 ft) from the bottom (B) at the same location. The Type 1 design velocities were obtained to form a baseline condition. Subsequent modifications to the model and the effect on the downstream channel velocity magnitude and direction can be compared with the Type 1 design.

Downstream velocities for flow conditions A, B, C, and D are shown as Plates 6-9. Average bottom velocities just downstream from the weir ranged from 4.9 ft/sec on the left side of the channel to 12.9 ft/sec just to the right of the channel center line. Surface velocities were much lower and better distributed (ranged from 2.5 to 3.1 ft/sec). Farther down the channel, the flow velocities were higher in the center of the channel, and the surface velocities were higher than the bottom velocities.

## **Type 2 Design**

Experiments were performed next to explore methods to route the overtopping flow into the stilling basin. With the Type 2 design, the overtopping flow would flow down the backside of the dam, then be routed to the stilling basin by a gutter channel (see Photo 13). From observations of the Type 1 existing design, an initial gutter width of 9.1 m (30 ft) was determined. Initially, only the right side gutter was installed. Once a suitable plan was developed, the left side would be installed and the total design documented.

The Type 2 design gutter channel is shown in Photos 13-14. Details of the Type 2 design are shown in Plate 10. At the top of the slope, the gutter is 6.1 m (20 ft) wide to allow for the roadway flow and to compensate for arch interference with the flow. At the toe of the slope, a paved apron was designed to allow the gutter flow to expand on the flat area. Riprap borders the apron and stilling basin to reduce erosion. Observations of the flow conditions with Condition D indicated that a barrier along the road was needed to prevent any flow from entering by the road.

For performance and evaluation purposes, the modification was subjected to the PMF for a duration of 1 hr prototype (7.75 min model). Results of the Type 2 design, Photo 15, show that the flow upon leaving the confined gutter section expanded more than estimated and eroded the riprap protection. Failure of the downstream section of the riprap was caused by wave action from the stilling basin.

### **Type 3 Design**

Because of failure of the riprap with the Type 2 design, the Type 3 design, Photo 16, was developed. This design expanded the concrete apron 30 ft and used larger riprap. Dimensions of the Type 3 design are shown in Plate 11. After subjecting the design to the PMF for 1 hr prototype (7.75 min model), damage because of flow expanding from the gutter and stilling basin wave action was still present, as shown in Photo 17.

### **Type 4 Design (right side)**

After evaluating the results of the concrete apron and riprap protection, the decision was made to use concrete to protect the apron from scour and wave action. The revised Type 4 apron, Figure 6, would begin at the end of the gutter wall and join the existing slab of the equipment building, then continue in a downstream direction until reaching the existing slope protection.

The Type 4 design also included modifications to the gutter channel, as shown in Figure 7 and Plates 12-13. At the upper end of the gutter (Plate 12), the first two arches were sealed to prevent disruption of the supercritical flow by the arch supports. The curved transition of the gutter replaces a sharp bend that caused disruption of the supercritical flow. At the lower end of the gutter (Plate 13), the flow expands onto an apron and into the stilling basin.

Flow conditions for the PMF are shown in Photos 18-19. In Photos 18-19, dye was added to the gutter flow to observe the flow from the gutter on the apron. Maximum depths in the right gutter channel are shown in Plate 14. Velocities, shown in Plate 15, were measured to show the magnitude and location of velocities on the concrete apron during PMF flow conditions. These velocities were measured approximately 1 ft above the ground surface.

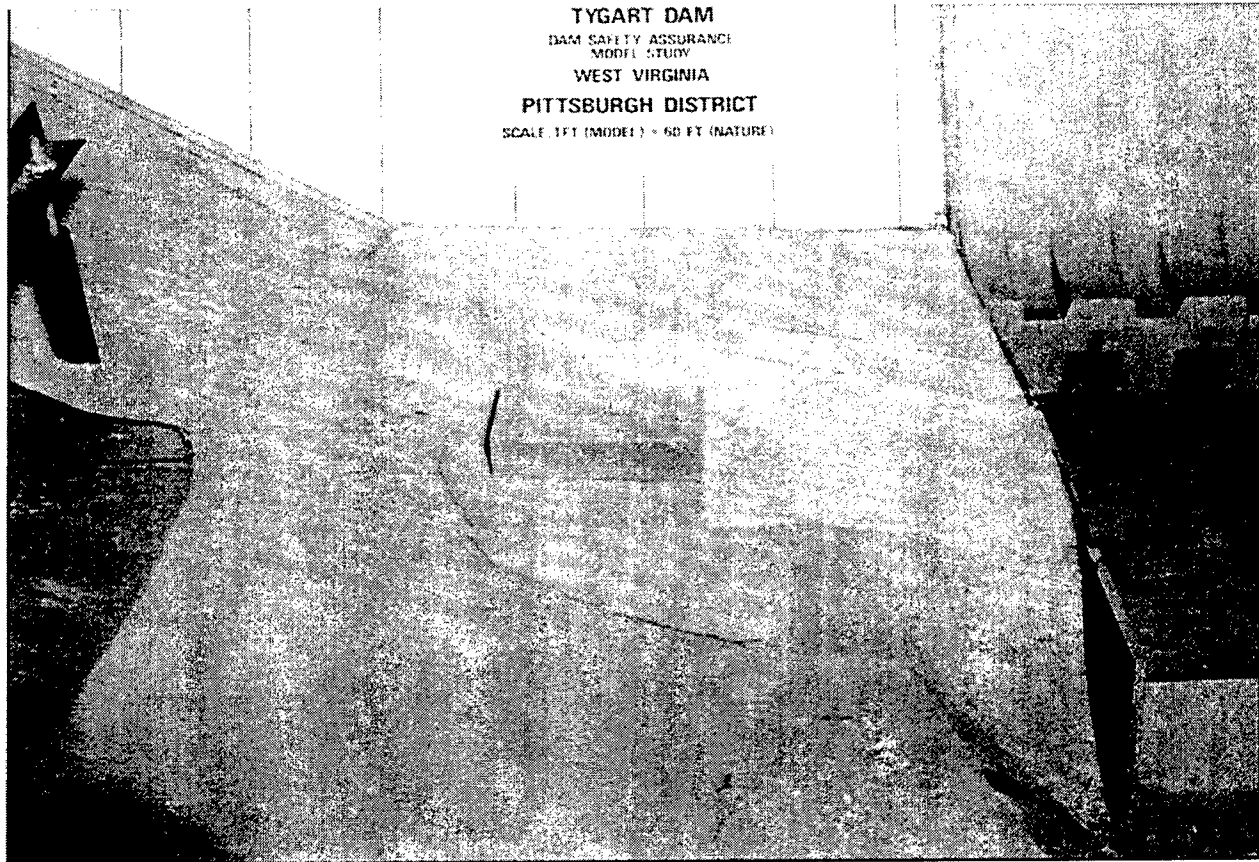


Figure 6. Type 4 design, paved apron, dry bed, right side

The model gutter channel was constructed of plastic-coated plywood with a vertical wall made of clear plastic. The Manning number for the model gutter channel is 0.009. Based upon Froudian scaling relationships, the corresponding prototype Manning's "n" value would be 0.018. The prototype gutter channel will be constructed of smooth concrete with an estimated Manning's "n" value of 0.012. Based upon these roughness differences, water-surface elevations in the gutter would be slightly lower and velocities would be slightly higher in the prototype when compared with the model results. However, in this study, the overtopping waters flow over the top of the dam, flow down the back side of the dam, then enter the gutter at supercritical velocity in a lateral direction. In the gutter, the flow builds up on the vertical outside wall and changes direction to flow down the gutter. Therefore, the water-surface elevations in the gutter are created by the intersecting downward flow from the gutter and lateral flow from the back of the dam. The resulting wave heights and flow buildup on the vertical outside wall created by this disturbance would be slightly greater in the prototype because of the differences in "n" values and velocity between the model and prototype.



Figure 7. Type 4 design, right side gutter channel and concrete apron, dry bed

### **Type 4 Design (left side)**

Based upon the success of the right side gutter channel, a similar modification was installed on the left side of the dam (see Figures 8 and 9). Dimensions of the left side gutter apron are shown in Plate 16. Because of differences in terrain, the slope of the gutter channels is not the same. The left side of the dam is not subject to personnel use as is the right side; therefore, an additional guide wall was used on the lower slope. Dimensions for the left side gutter and apron are shown in Plate 17. Maximum water depths in the left gutter channel are shown in Plate 18. Velocities on the left concrete apron are shown in Plate 19 and provide the location and magnitude of velocities created by PMF conditions. Flow conditions on the left side are shown in Photo 19.

### **Combined Type 4**

With both gutter channels installed, the Tygart Dam and spillway were able to satisfactorily pass the PMF. The overall PMF conditions are shown in Figure 10. Velocities in the downstream channel were recorded to examine

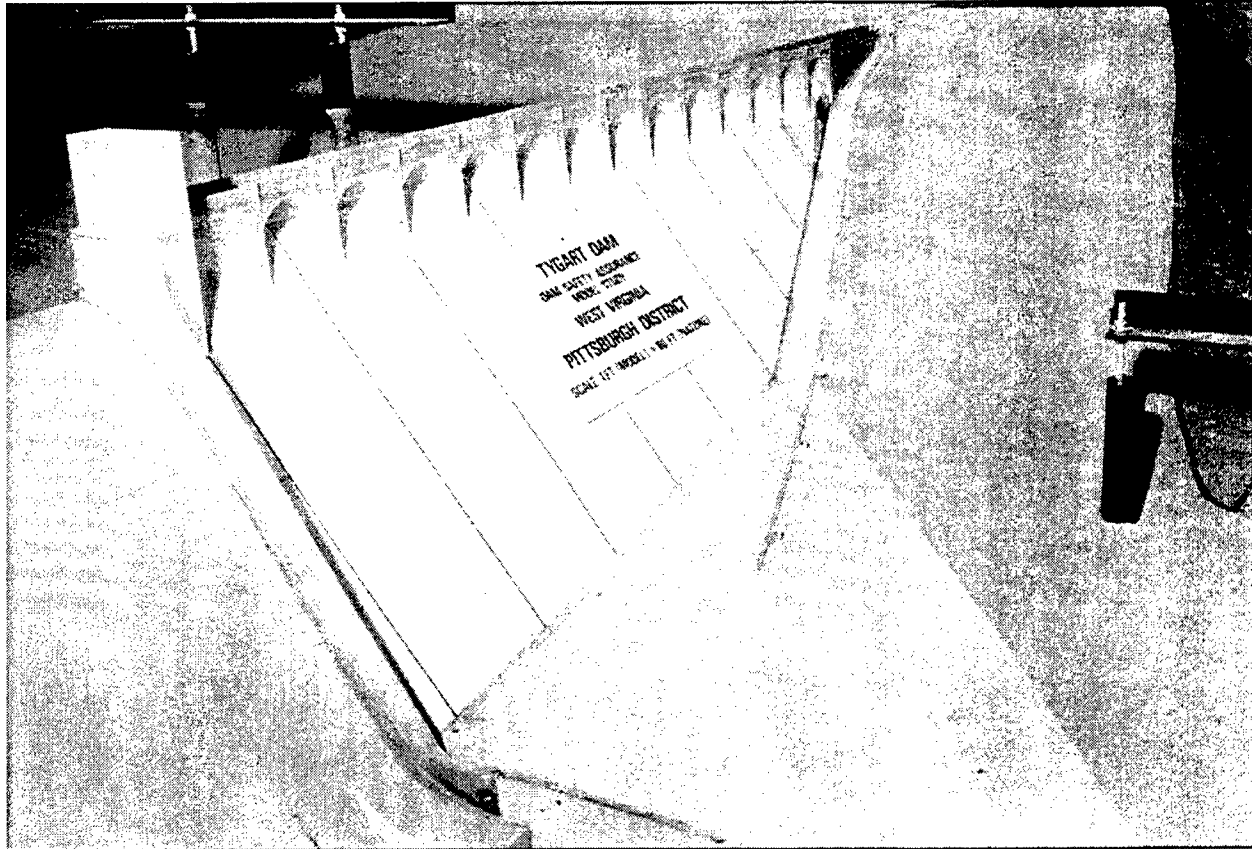


Figure 8. Type 4 design, left side gutter channel, guide wall, and paved apron, dry bed

changes in velocity caused by the gutter channel modifications. Comparing the downstream bottom velocities of the Type 4 (Plate 20) with those of the Type 1 (Plate 9) showed little or no change in the channel bed because of the gutter and apron modification.

## Stilling Basin

The stilling basin, although originally designed for 6,089 cu m/sec (215,000 cfs), performed adequately for the increased discharge required by the PMF. Problems in stilling basin performance were observed because of the wave action spilling over the training walls. Ordinarily, the wave action could be contained by increased wall height. However, increasing the wall height would not allow the gutter flow to enter the stilling basin. Structural factors were also a factor when considering the age of the structure and the extensive modifications required to construct higher training walls. Adding more length to the stilling basin and redesigning the stilling weir were suggested during the model study as ways of reducing wave height. However, the model results showed that the stilling weir was still containing the hydraulic jump, thus making such modifications not necessary when considering the costs involved.

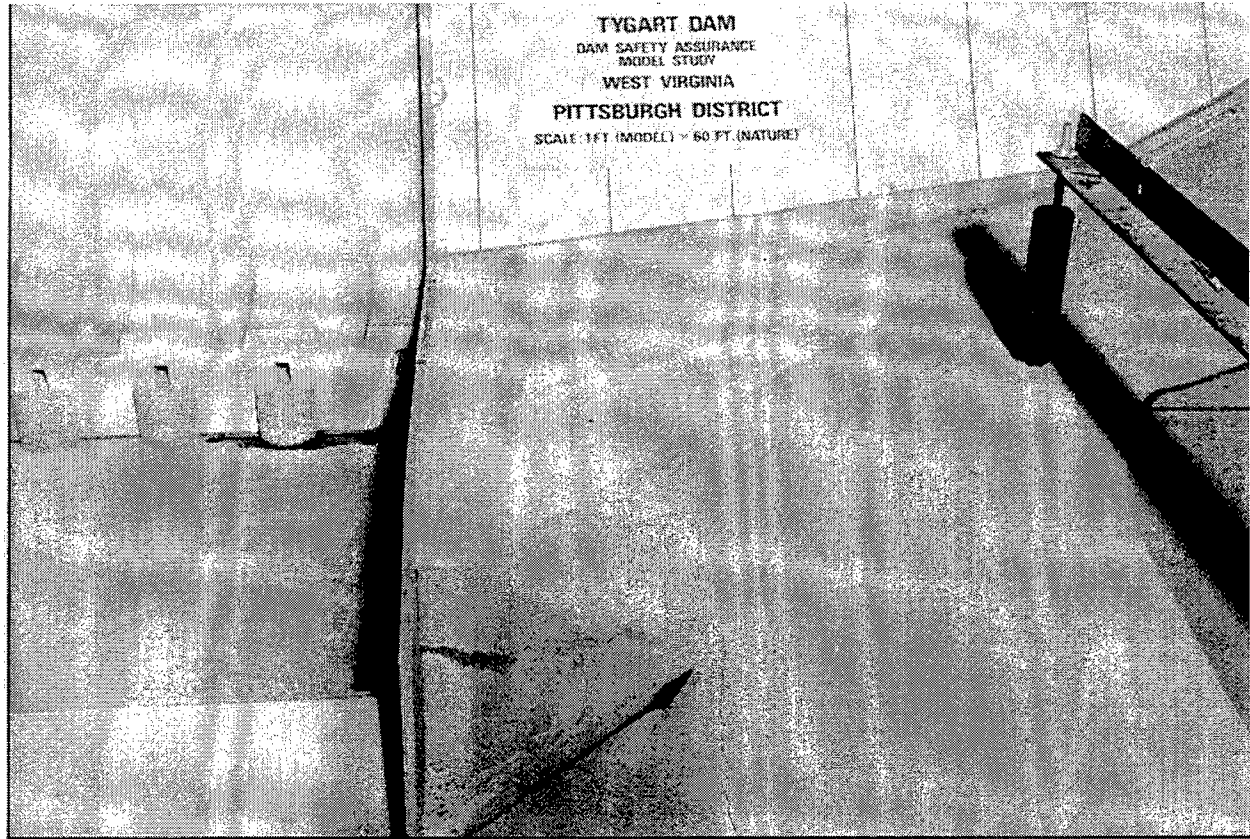


Figure 9. Type 4 design, left side guide wall and paved apron, dry bed

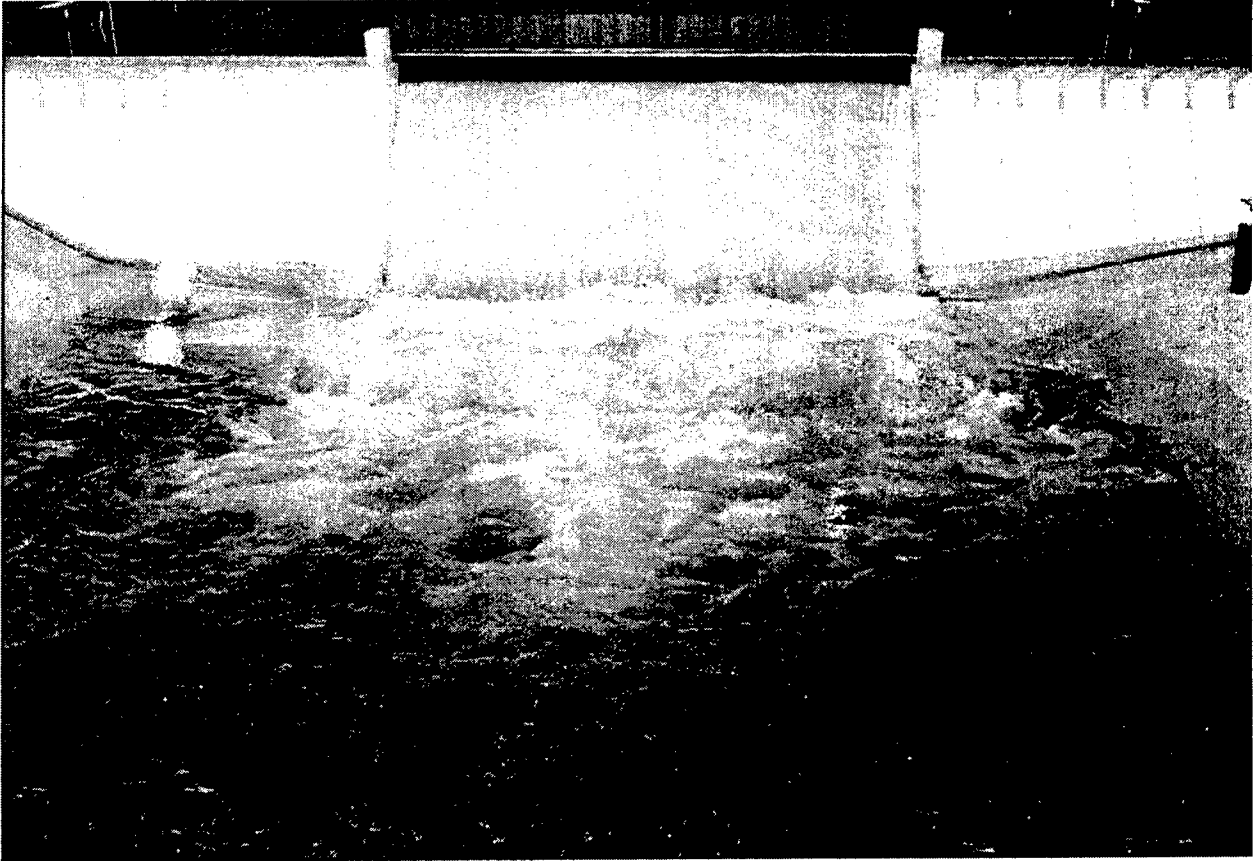


Figure 10. Type 4 design, PMF flow ( $Q = 373,120$  cfs; Pool = el 1196.95; TW el = 1031.5)

## 4 Conclusions and Recommendations

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Based upon the results of the model study, the Tygart Dam and spillway can safely pass the revised PMF with minimal modifications. While other methods of increasing the capacity of the structure were considered, such as raising the dam or lowering the spillway crest, the overtopping plan proved to be the most cost effective.

The overtopping plan allows the floodwaters to come over the dam. On the downstream side of the dam, gutter channels are constructed to catch and convey the overtopping flow down the side slope. Some additional freeboard should be added to the gutter walls to compensate for the increased water-surface elevations expected to occur in the prototype. A lip on the gutter wall would be beneficial to ensure that all the flow overtopping the dam is confined to the gutter channel. Without these gutter channels, scour on the back side of the dam would undermine the structure, which would lead to failure. The gutter channels direct the flow into concrete aprons that spread the flow into the stilling basin.

The increased discharge of the revised PMF was initially thought to exceed the capacity of the stilling basin. Model study experiments demonstrated that the existing stilling basin with the weir at the downstream end does not allow the hydraulic jump to sweep out. However, the wave action created in the stilling basin does pour over the stilling basin walls. Without adequate protection on the overbank, this wave action would cause scour. These scour-prone areas are protected by extending the concrete apron to cover these areas.

The downstream velocities showed little or no change because of the recommended modifications. Velocities of up to 18.6 ft/sec were observed 200 ft downstream of the stilling weir. These high velocities are caused by the stilling weir and the secondary jump. Velocities of this magnitude could cause scour of the downstream channel. However, in this channel, the bed material is hard rock and able to withstand velocities of this magnitude. Some movement of material is to be expected for the PMF condition; however, severe scour or undermining is not expected to occur. Because of the channel bed material, these velocities were not severe enough to require modification to the downstream channel.

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Therefore, the overtopping plan with gutter channels and concrete aprons is the recommended design.

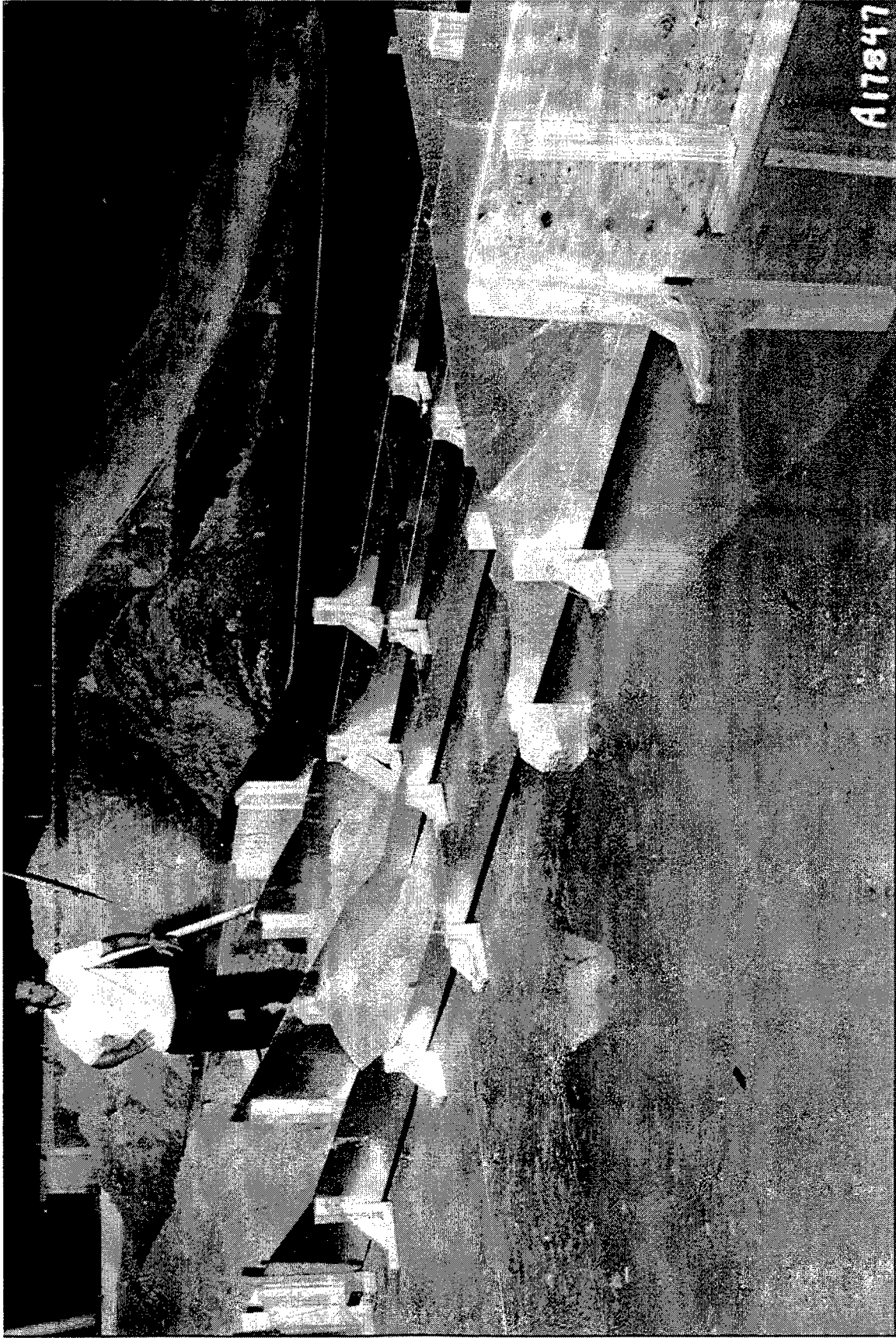


Photo 1. Tygart model construction

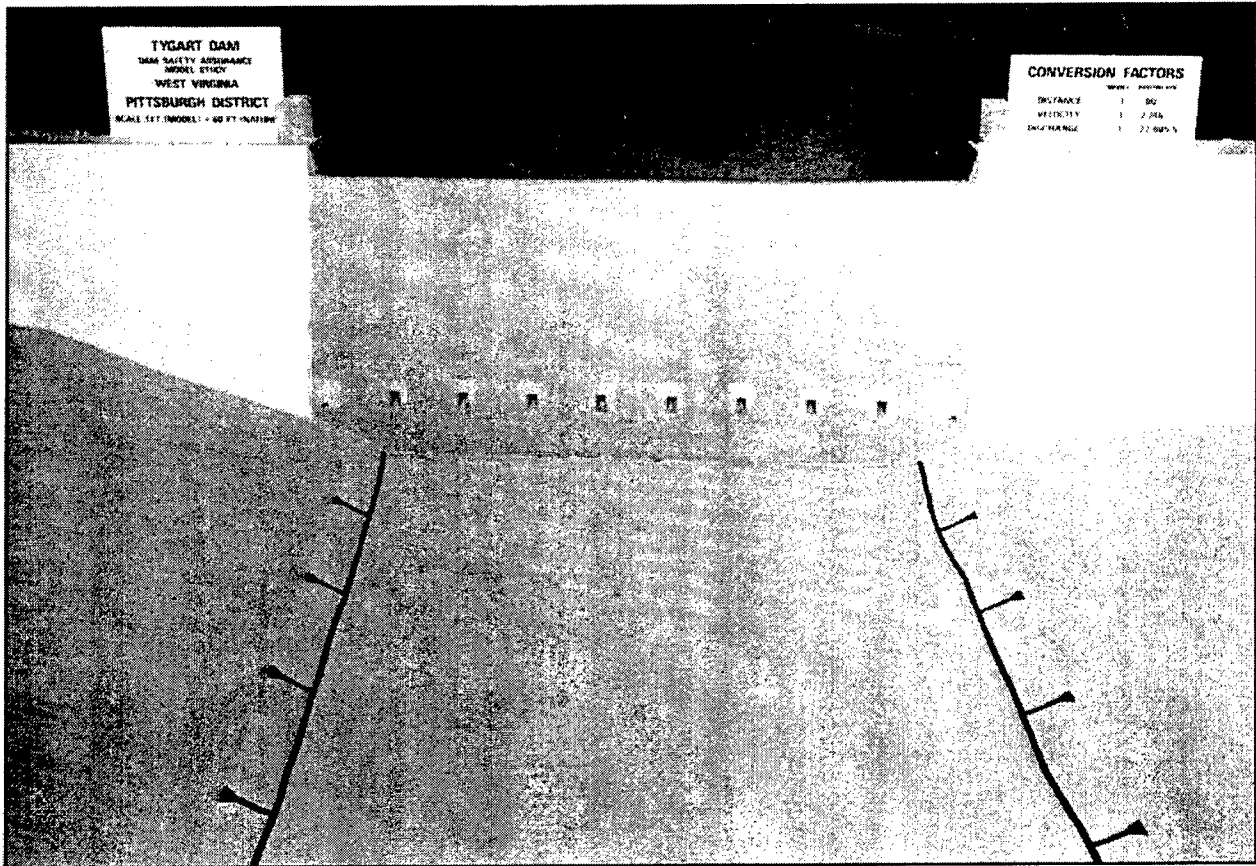


Photo 2. Tygart spillway, upstream view, dry bed

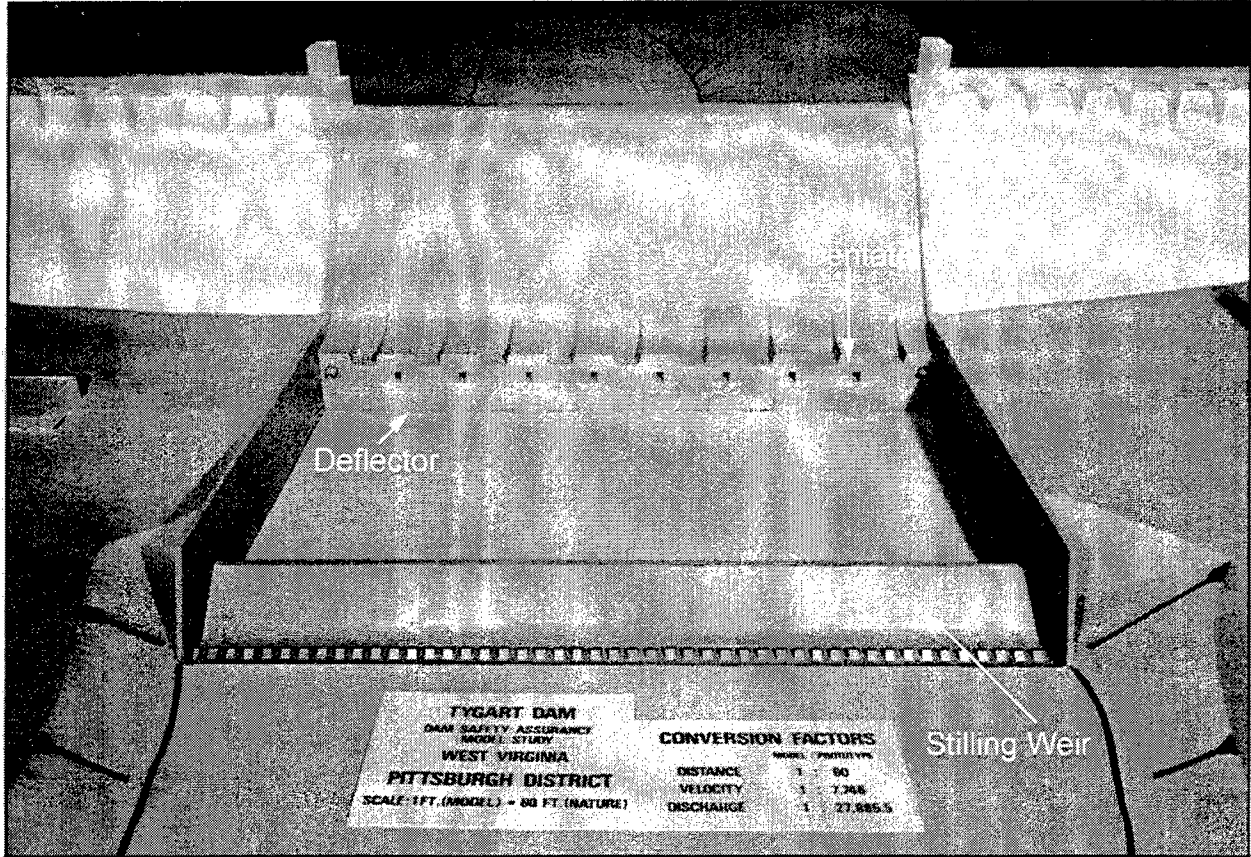


Photo 3. Tygart spillway and stilling basin, downstream view, dry bed

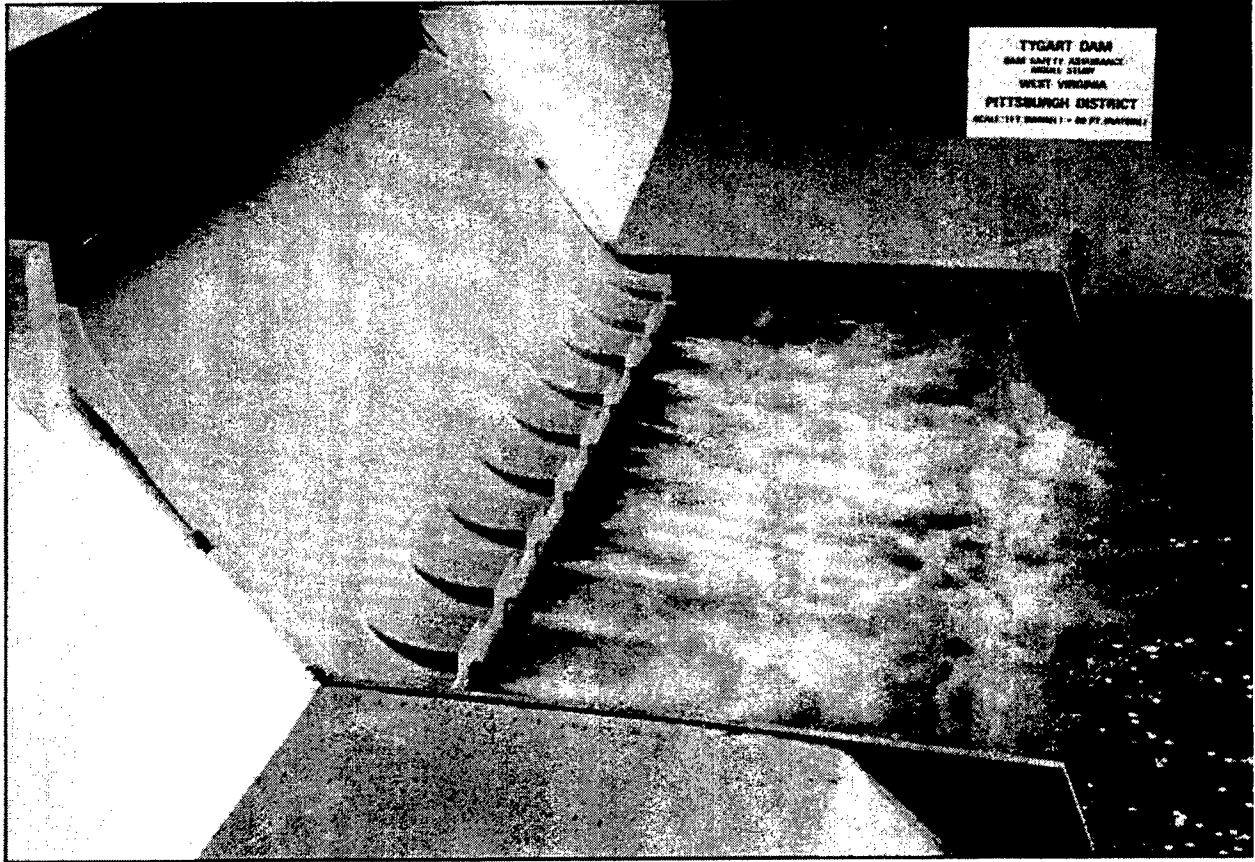


Photo 4. Tygart stilling basin flow conditions, sluice flow only ( $Q = 41,580$  cfs; Pool = el 1167.0; TW el = 987.0)

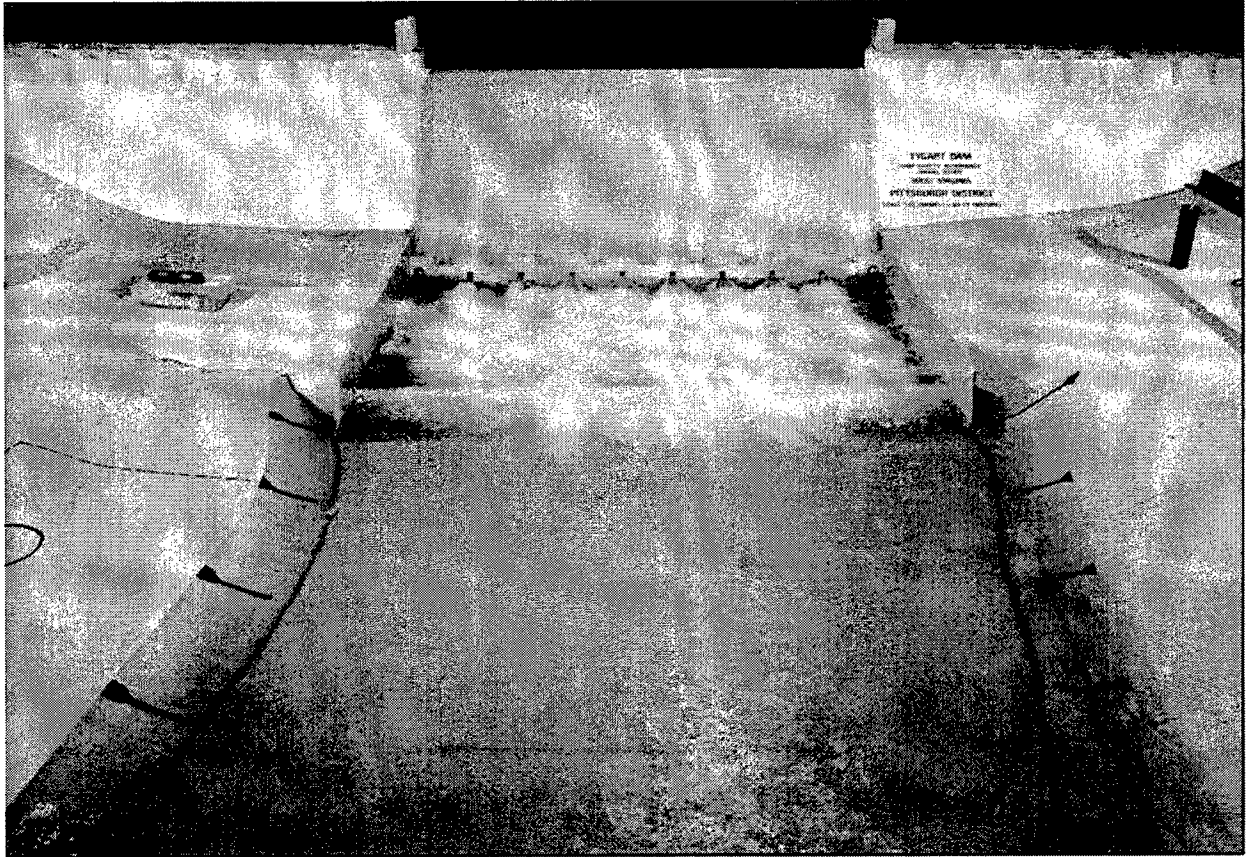


Photo 5. Tygart stilling basin flow conditions, sluice flow only ( $Q = 41,580$  cfs; Pool = el 1167.0; TW el = 987.0)

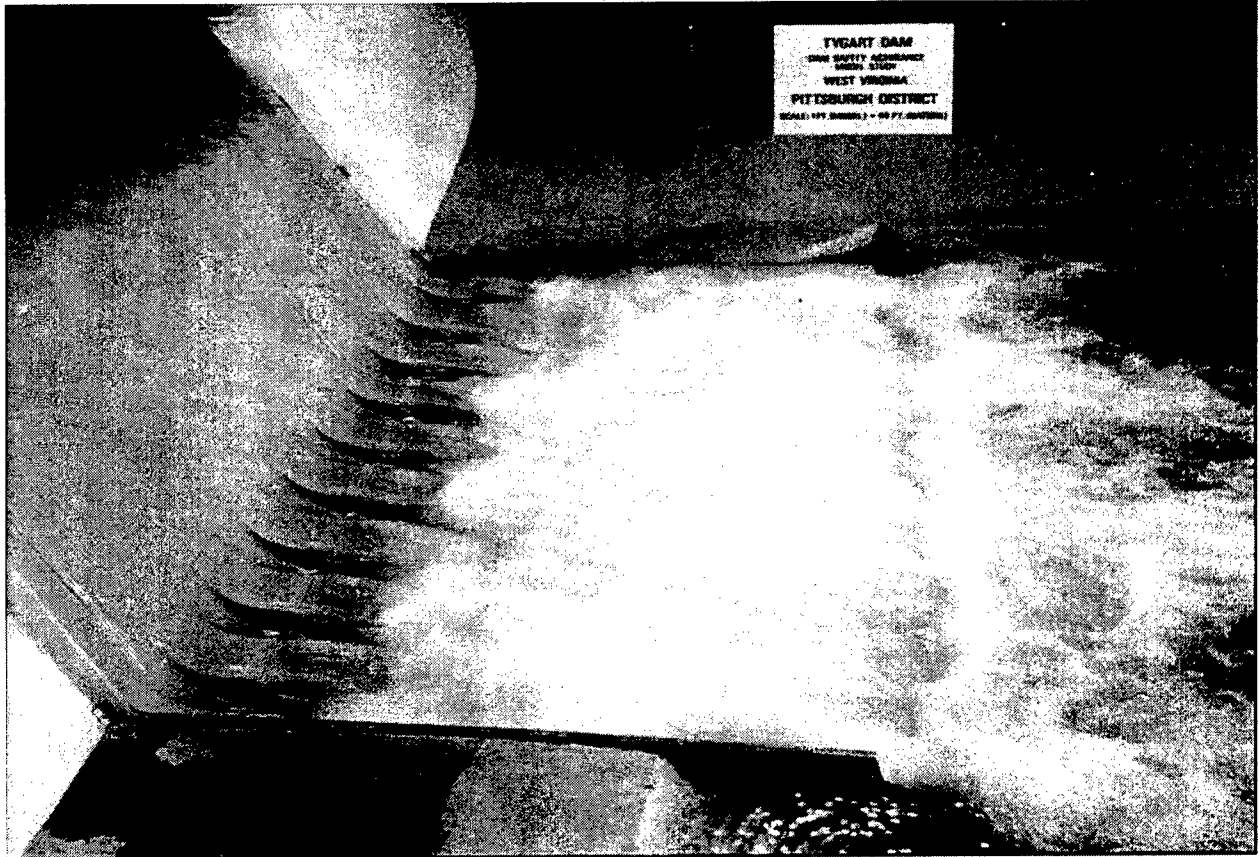


Photo 6. Tygart stilling basin flow conditions, spillway and sluice flow ( $Q = 250,780$  cfs; Pool = el 1190.0; TW el = 1020.0)

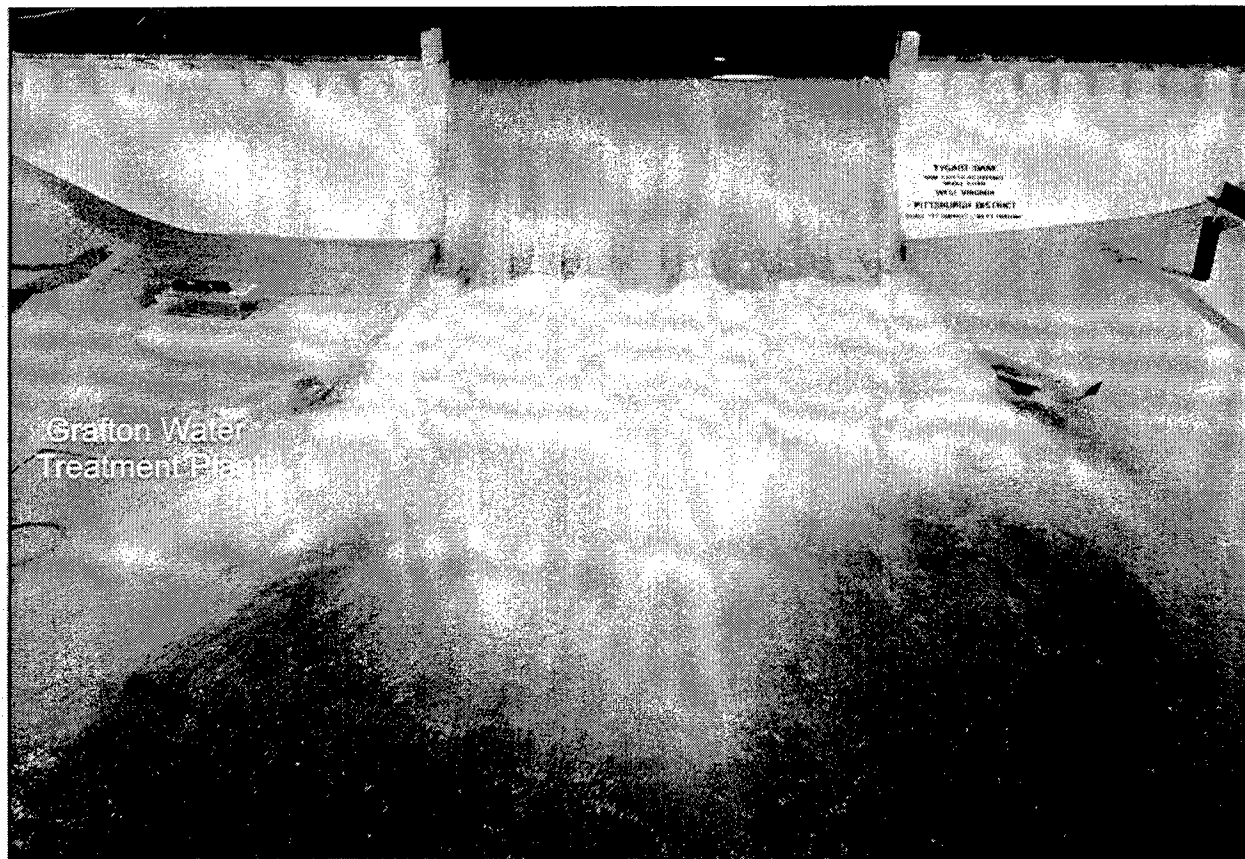


Photo 7. Tygart stilling basin flow conditions, spillway and sluice flow ( $Q = 250,780$  cfs; Pool = el 1190.0; TW el = 1020.0)

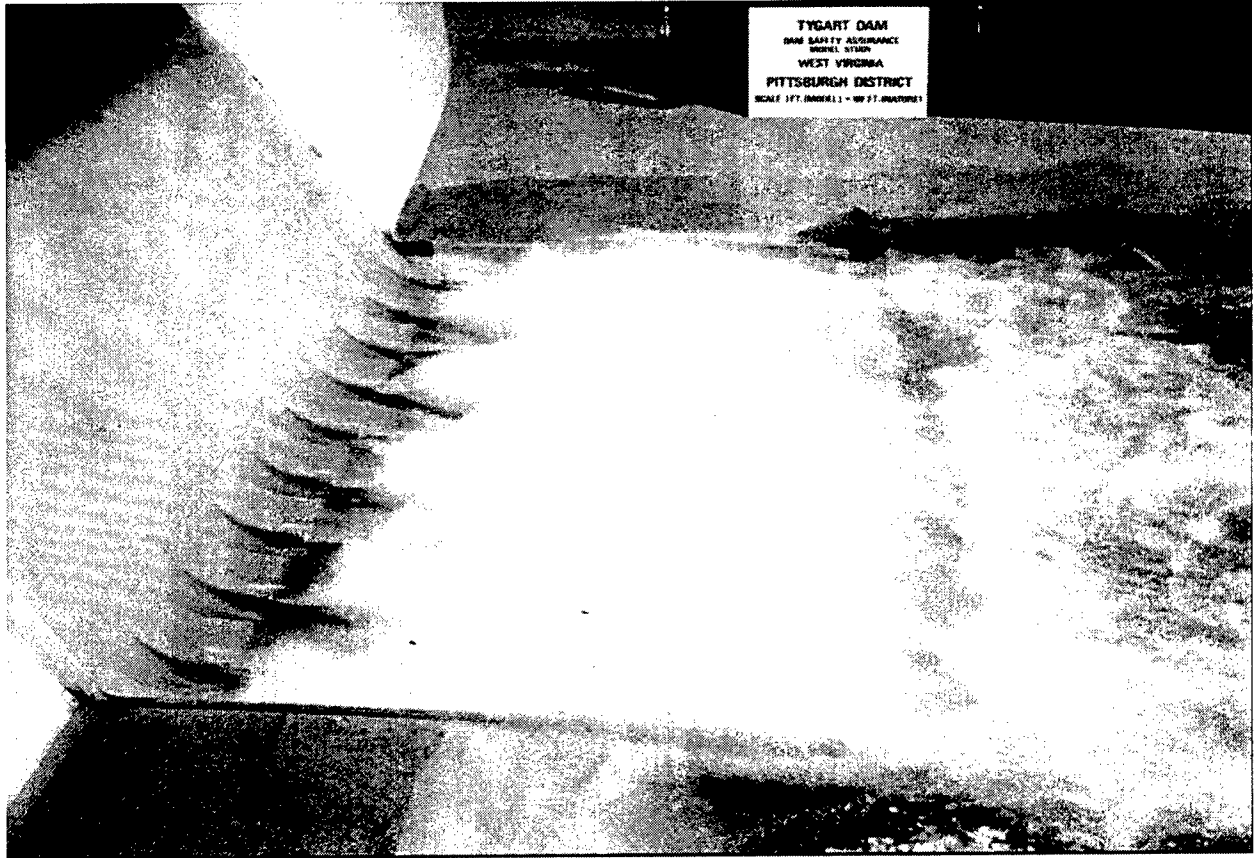


Photo 8. Tygart stilling basin flow conditions, spillway and sluice flow (Q = 319,080 cfs; Pool = el 1194.25; TW el = 1026.5)

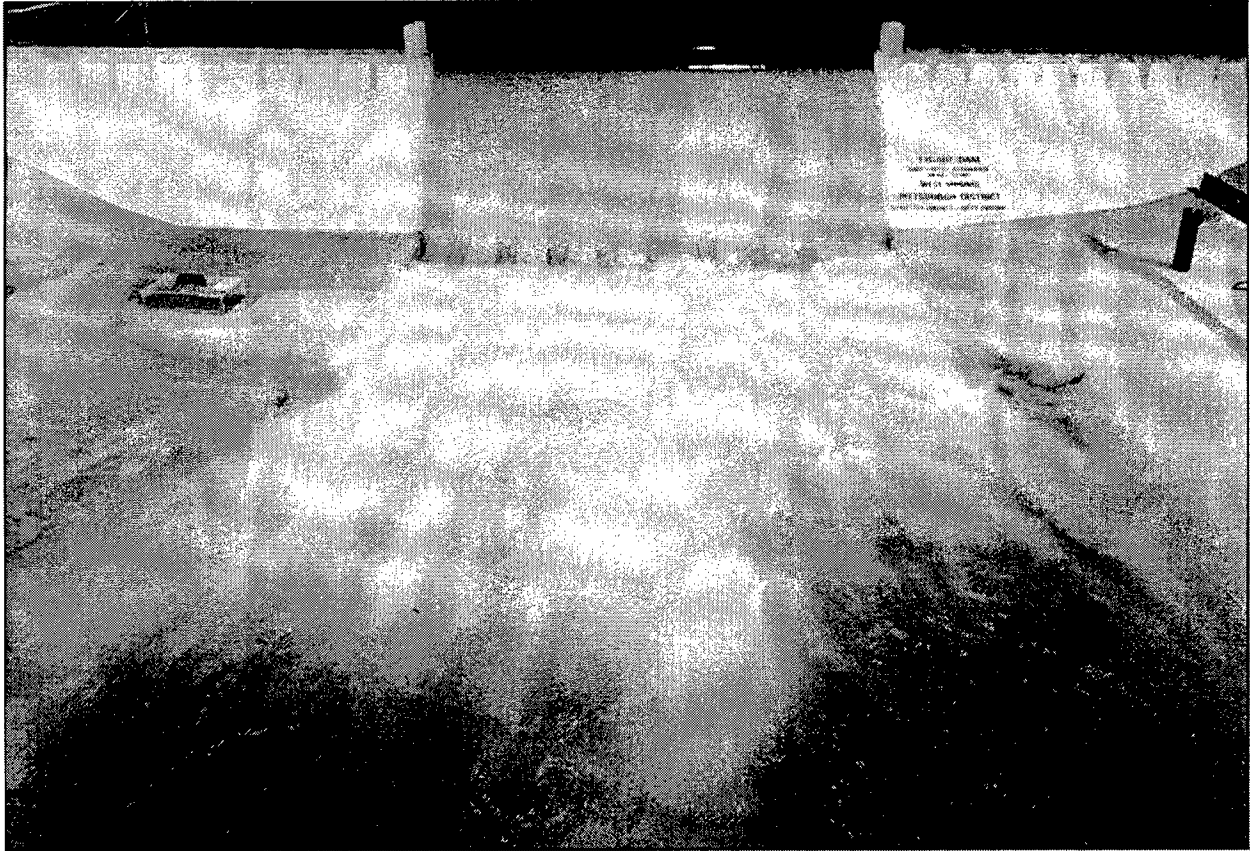


Photo 9. Tygart stilling basin flow conditions, spillway and sluice flow (Q = 319,080 cfs;  
Pool = el 1194.25; TW el = 1026.5)

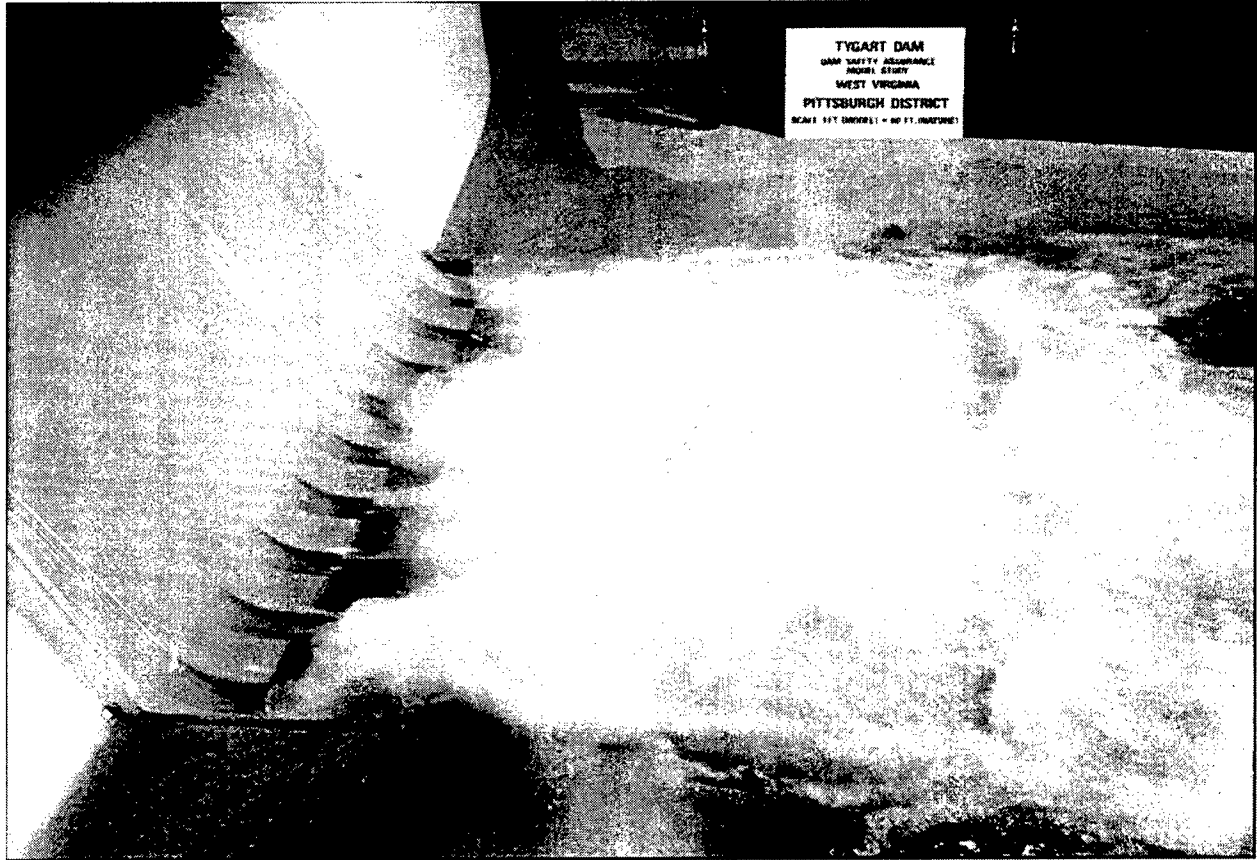


Photo 10. Tygart stilling basin flow conditions, PMF overtopping, spillway and sluice flow (Q = 373,120 cfs; Pool = el 1196.95; TW el = 1031.5)

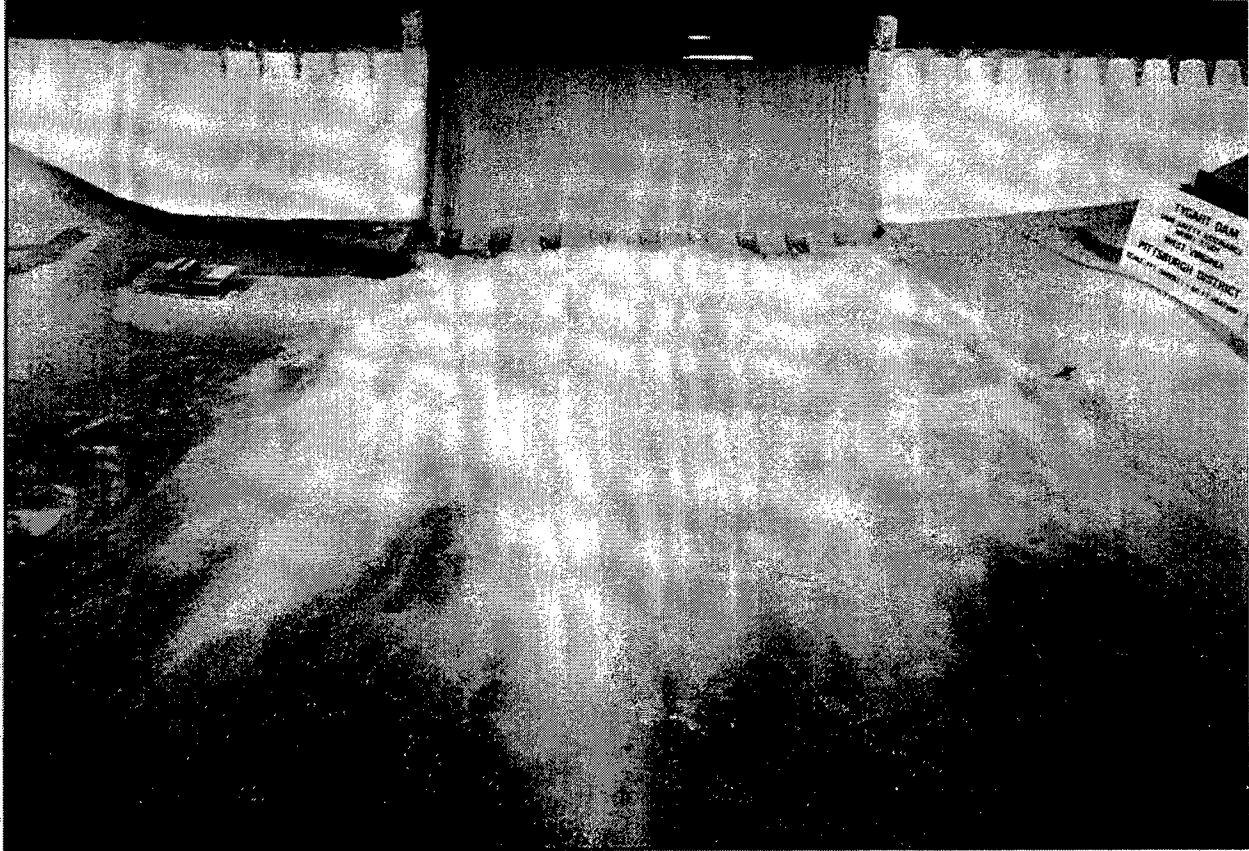


Photo 11. Tygart stilling basin flow conditions, PMF overtopping, spillway and sluice flow  
( $Q = 373,120$  cfs; Pool = el 1196.95; TW el = 1031.5)

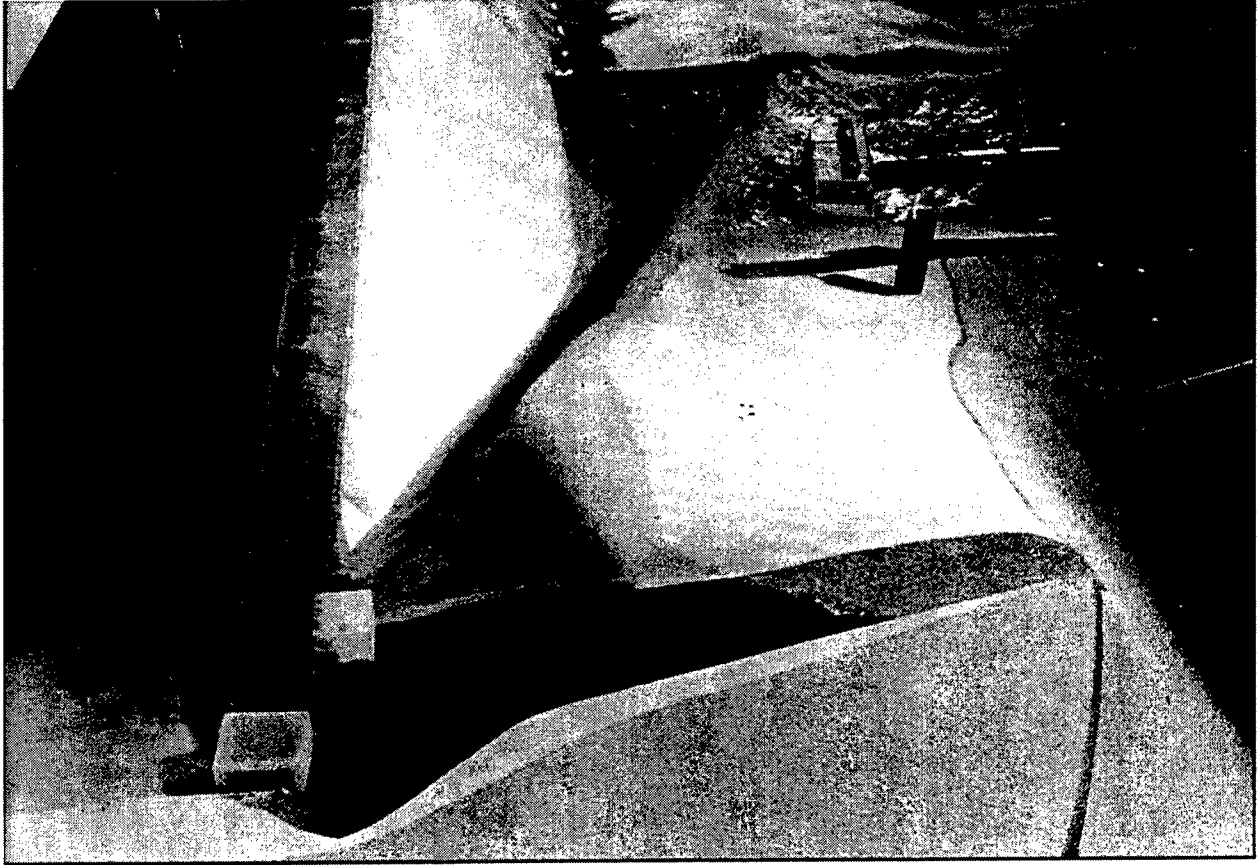


Photo 12. Tygart overflow on right side of dam, PMF flow( $Q = 373,120$  cfs; Pool = el 1196.5; TW el = 1031.5)

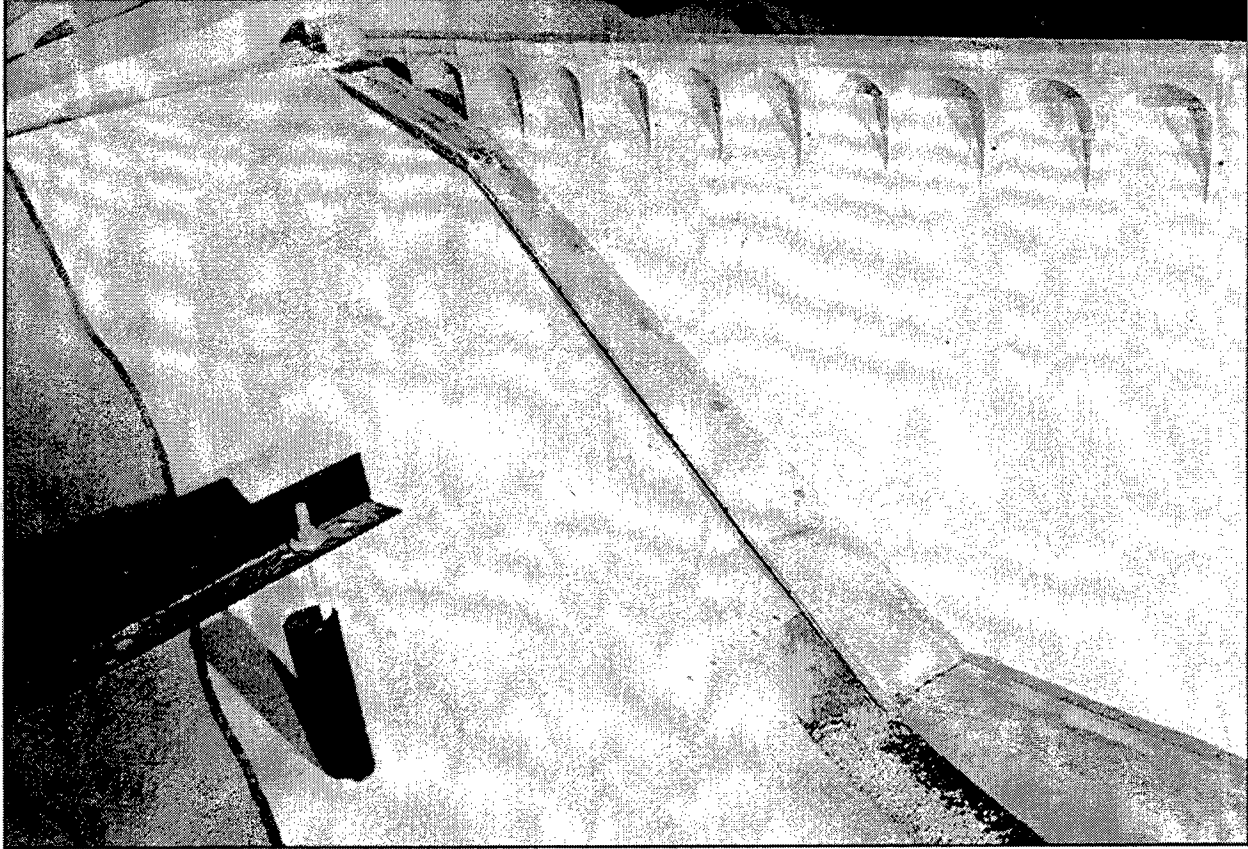


Photo 13. Type 2 design, gutter channel, dry bed, right side of dam

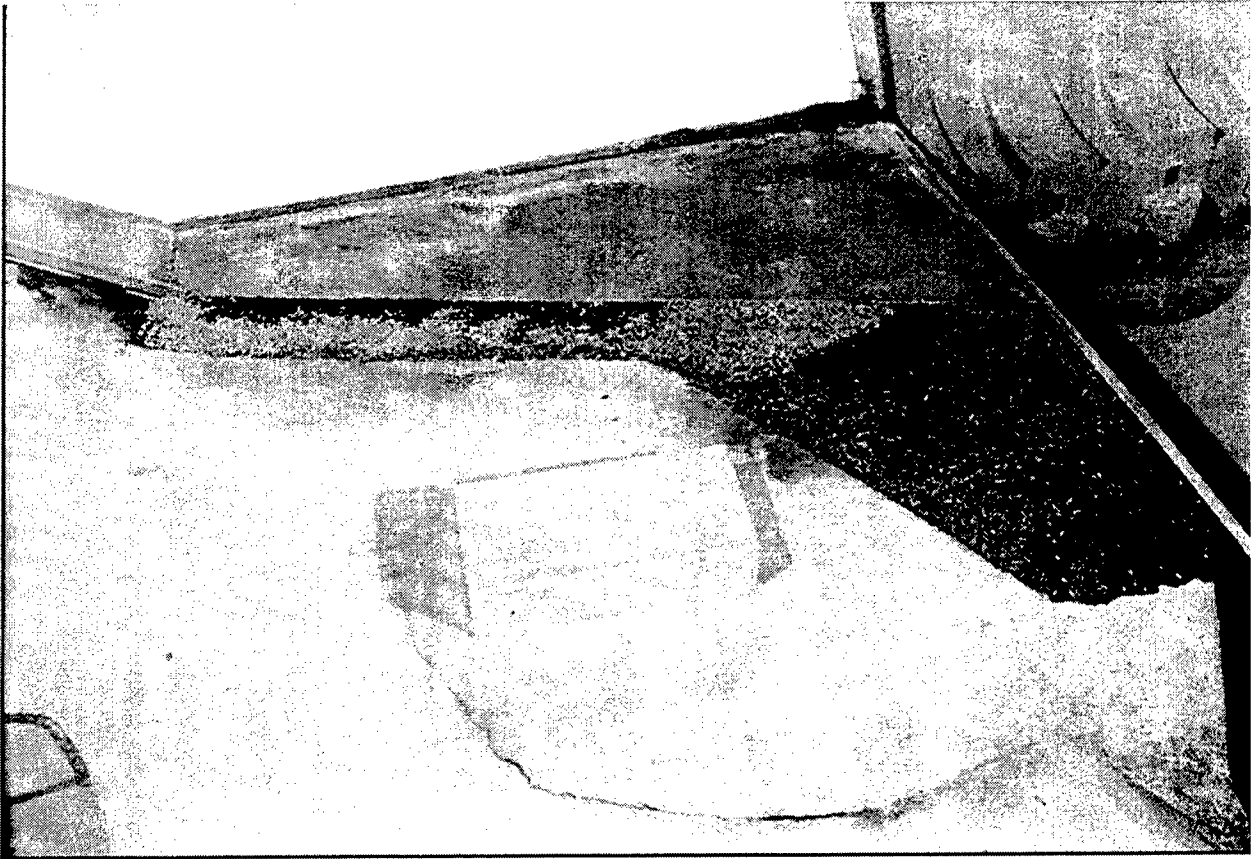


Photo 14. Type 2 design, apron and riprap, dry bed, right side of dam

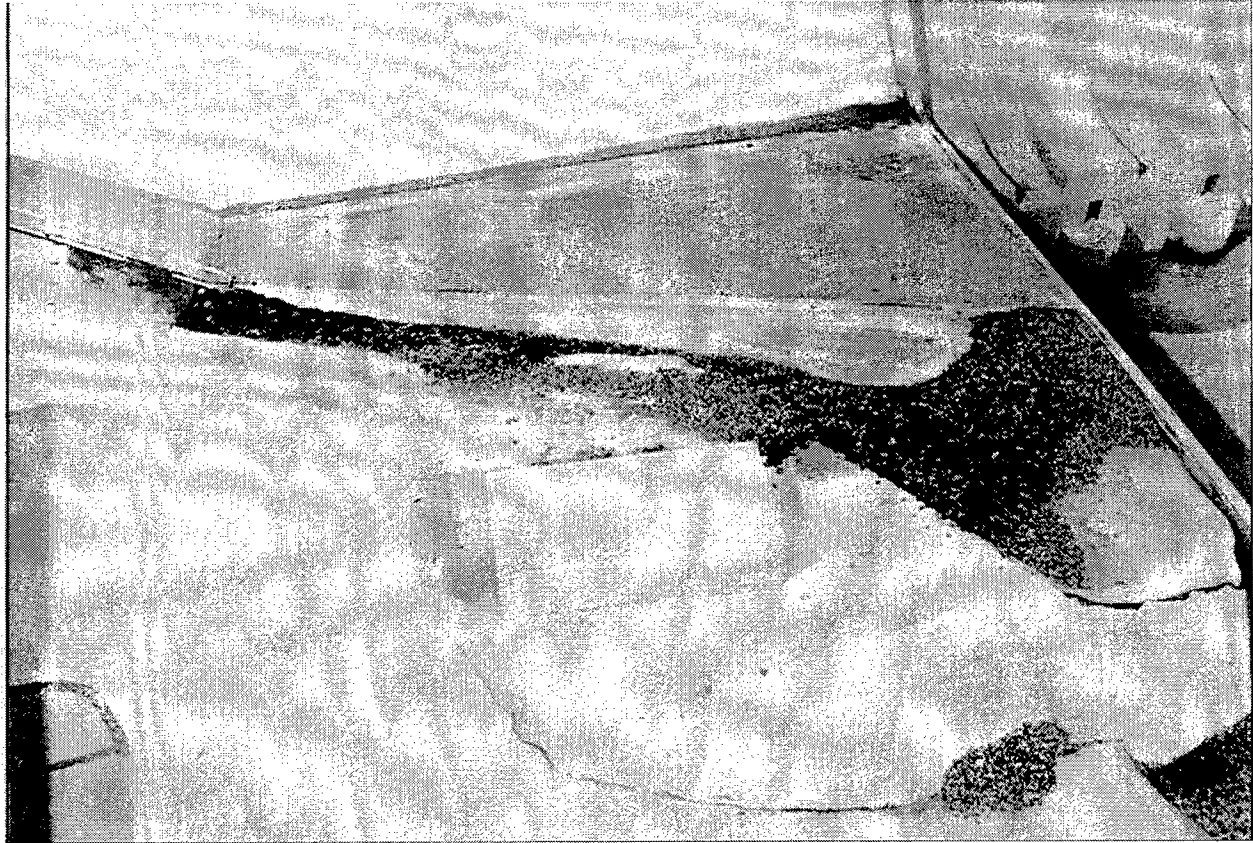


Photo 15. Type 2 design, damage resulting from PMF flow for 1 hr (proto), right side of dam

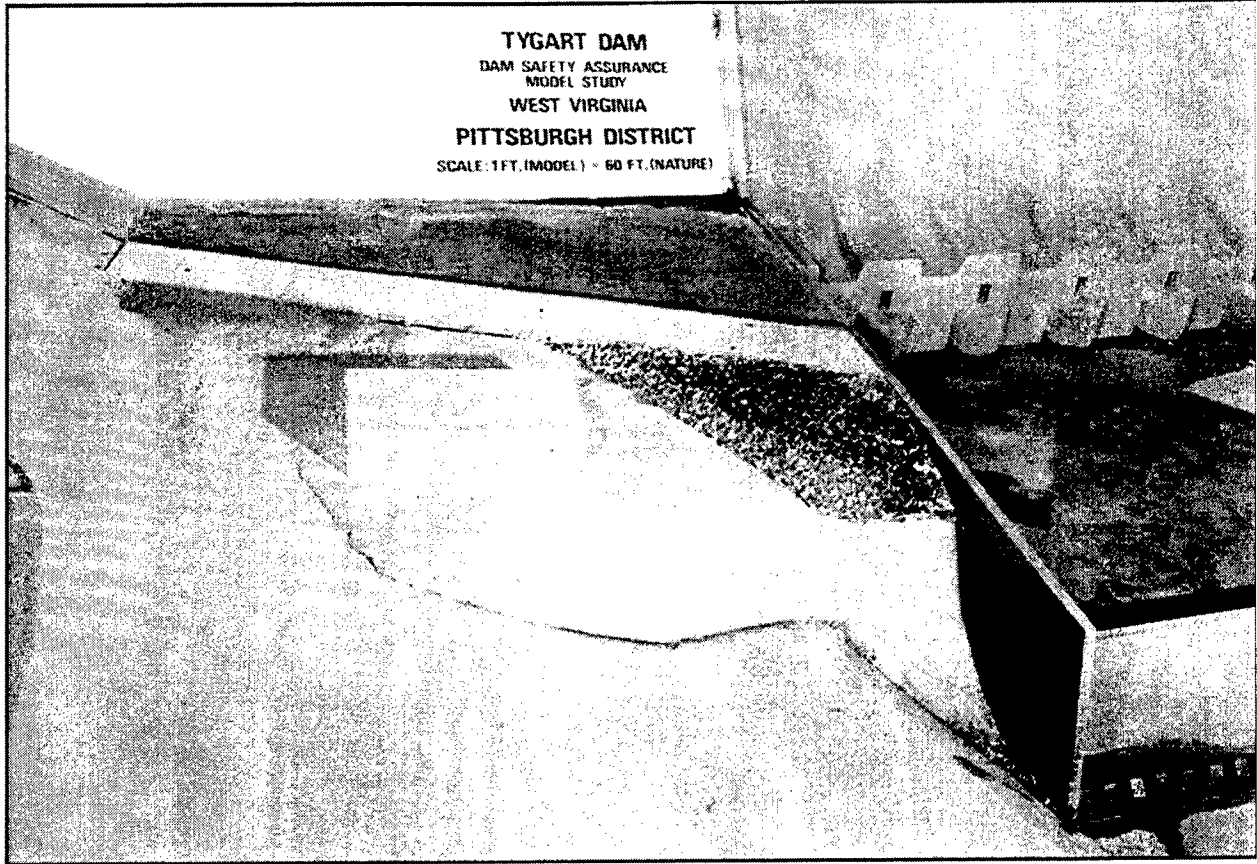


Photo 16. Type 3 design, apron and riprap, dry bed, right side of dam

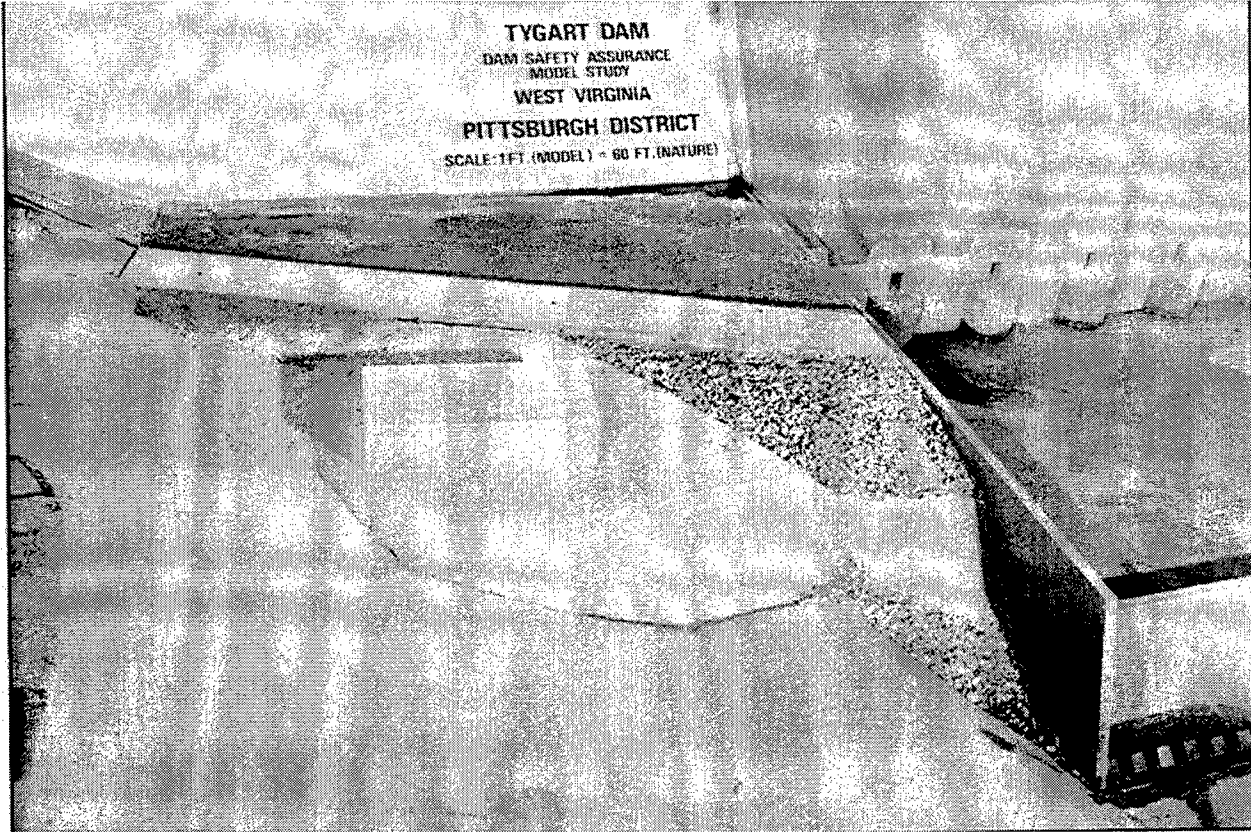


Photo 17. Type 3 design, damage resulting from PMF flow for 1 hr (proto), right side of dam

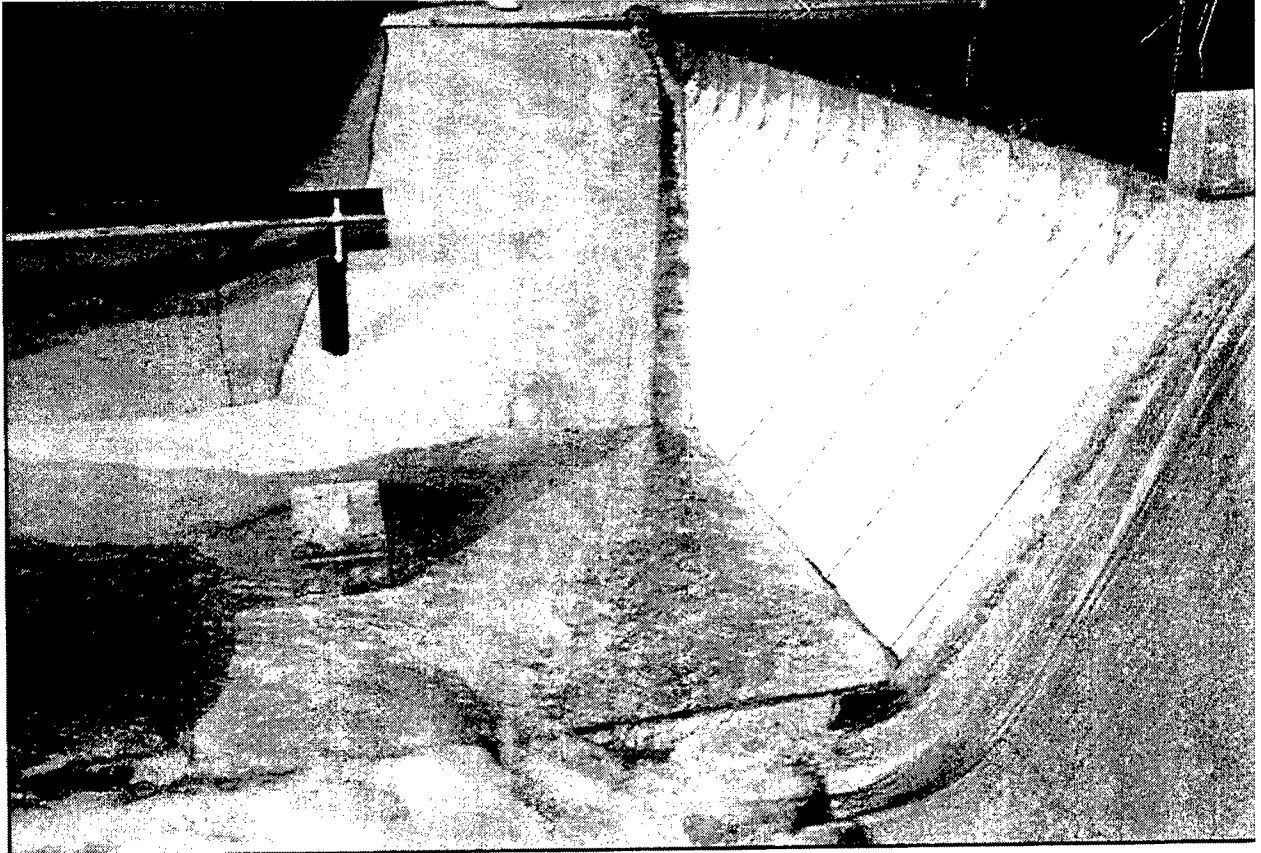


Photo 18. Type 4 design, PMF overflow on right side of dam ( $Q = 373,120$  cfs; Pool = el 1196.95; TW el = 1031.5)

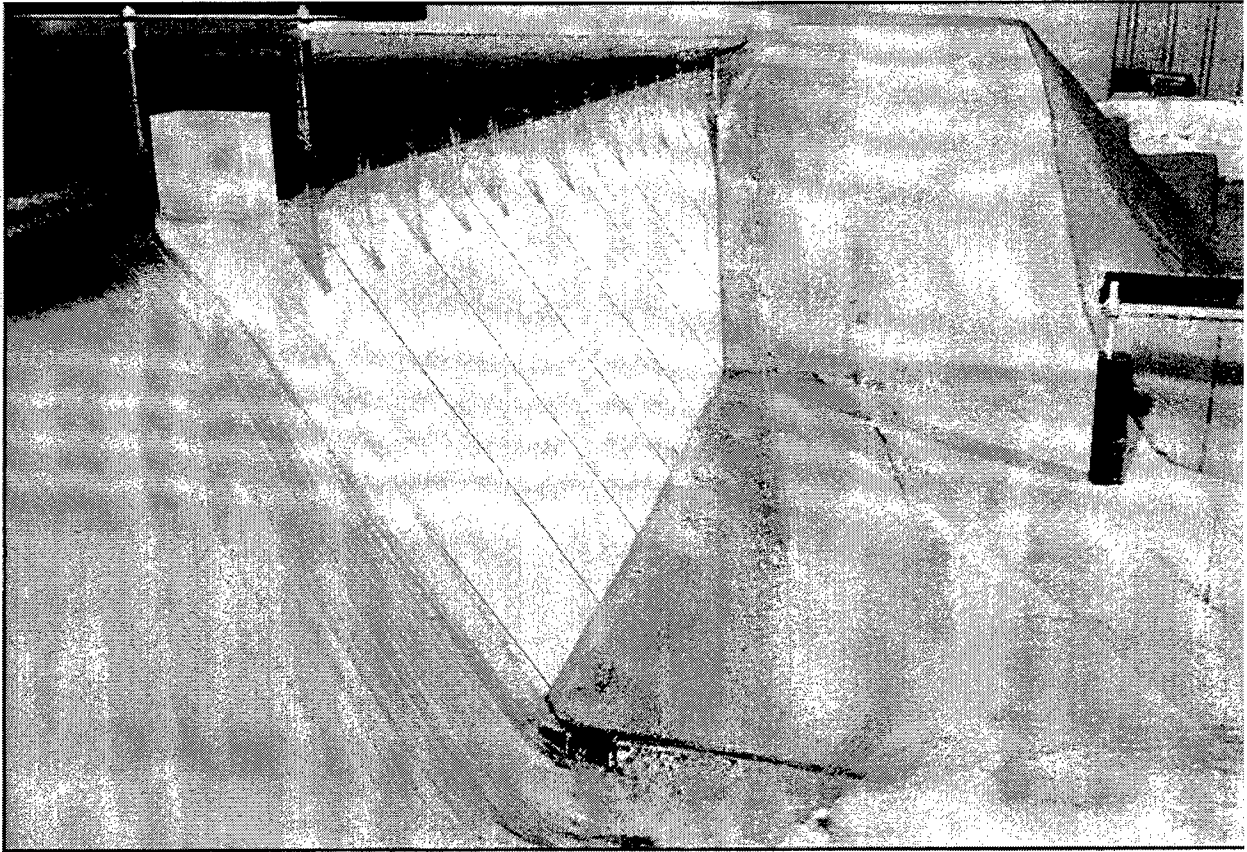
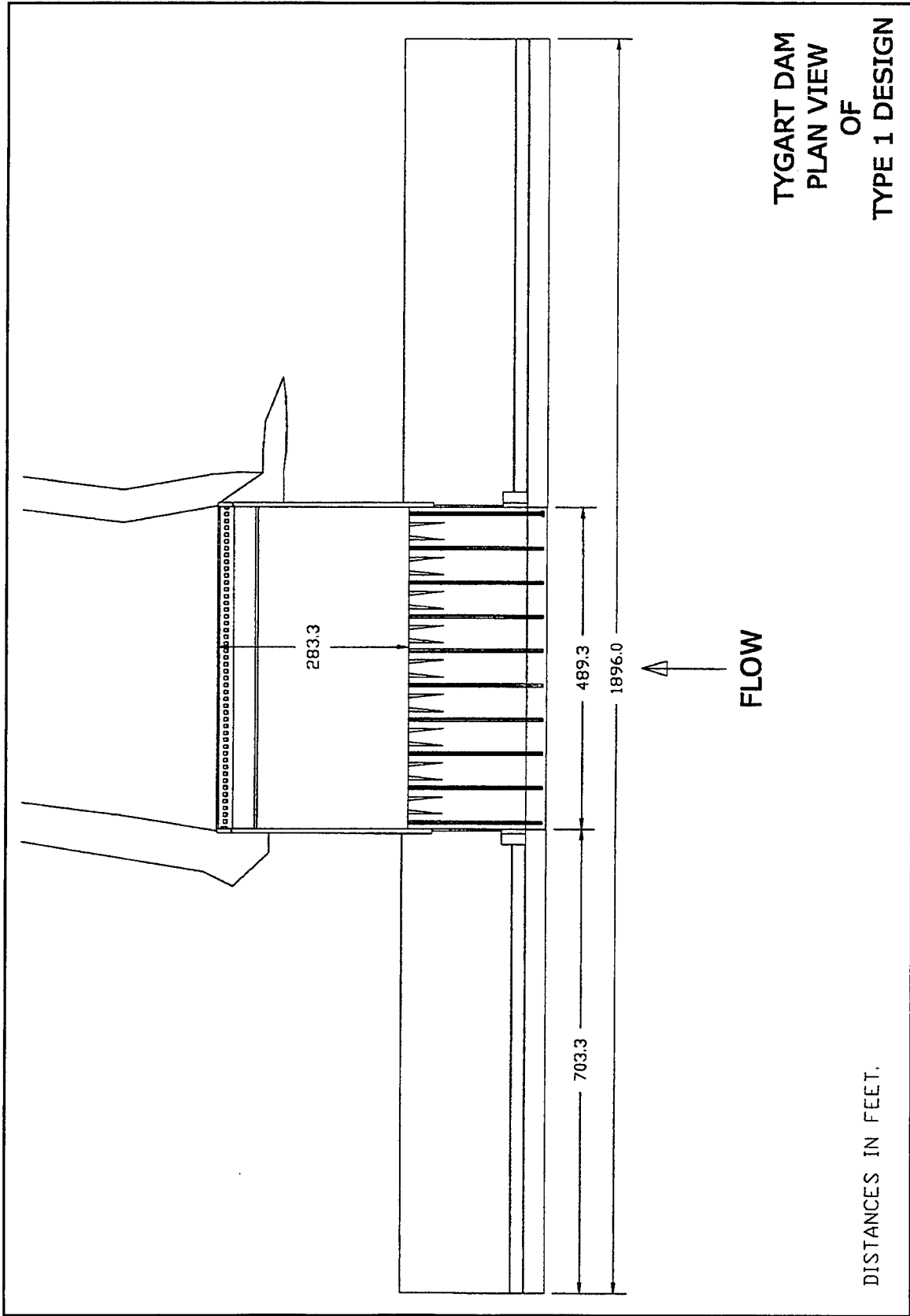


Photo 19. Type 4 design, PMF overflow on left side of dam( $Q = 373,120$  cfs; Pool = el 1196.95; TW el = 1031.5)





TYGART DAM  
 PLAN VIEW  
 OF  
 TYPE 1 DESIGN

DISTANCES IN FEET.

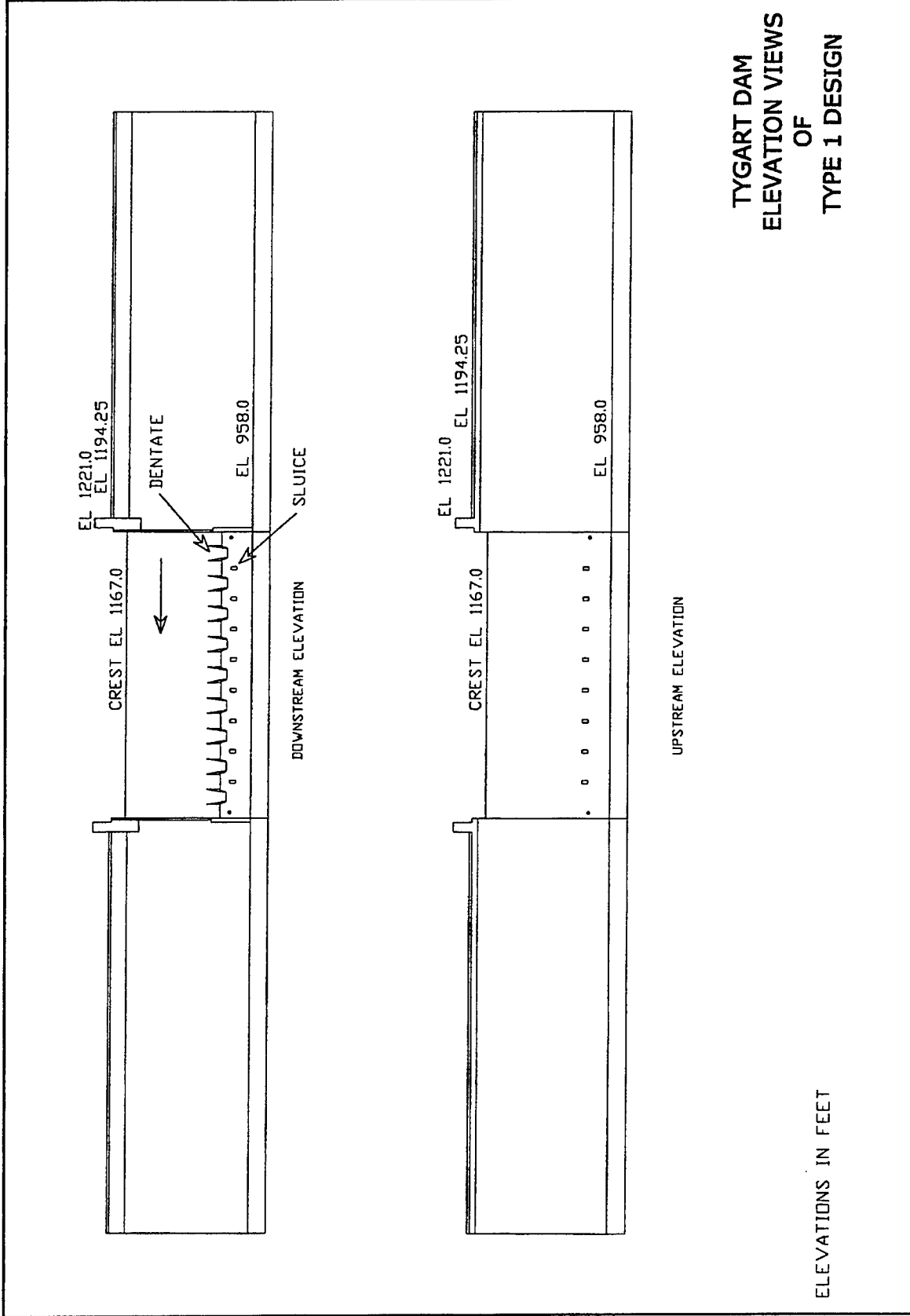
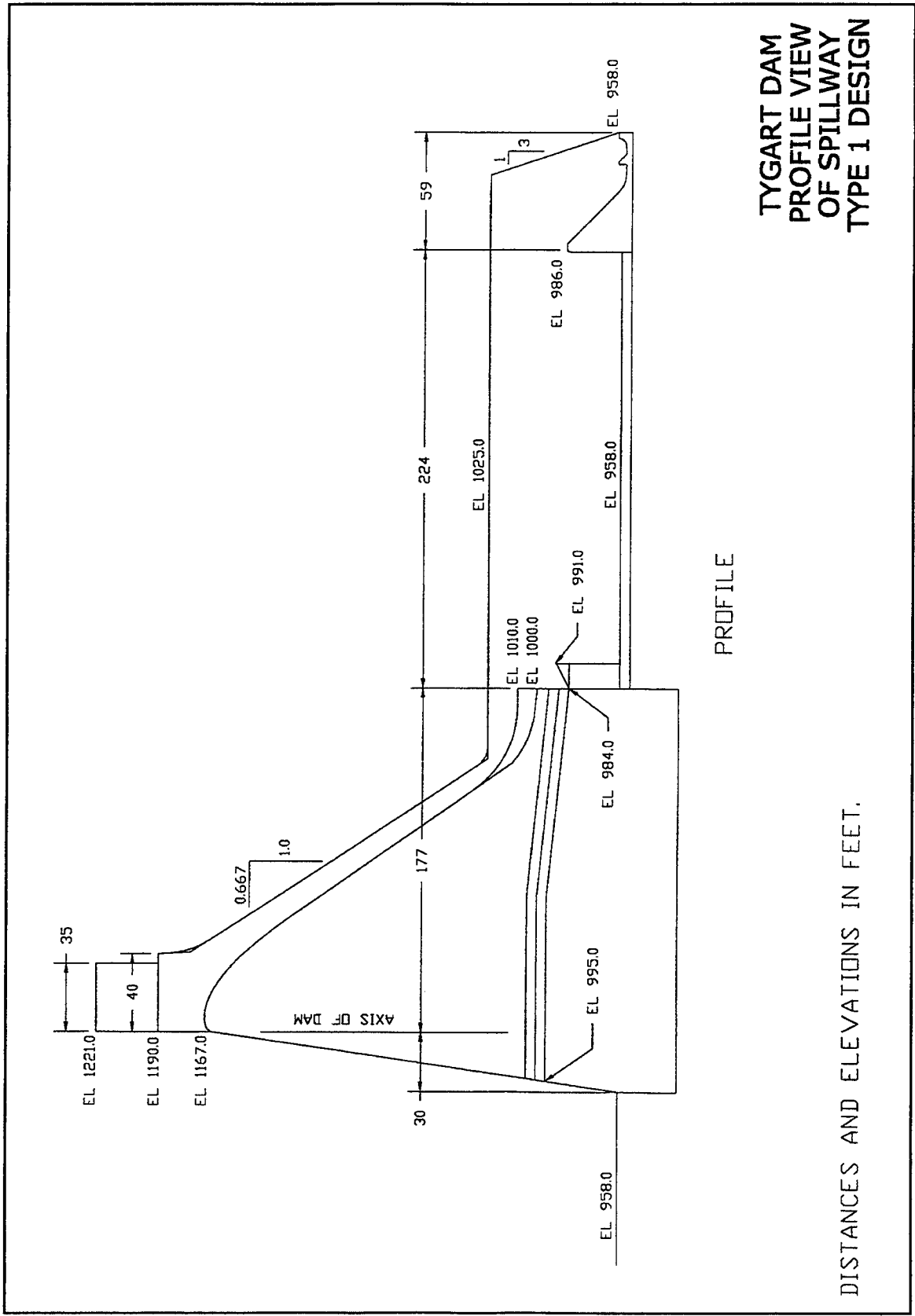


Plate 2



**TYGART DAM  
 PROFILE VIEW  
 OF SPILLWAY  
 TYPE 1 DESIGN**

PROFILE

DISTANCES AND ELEVATIONS IN FEET.

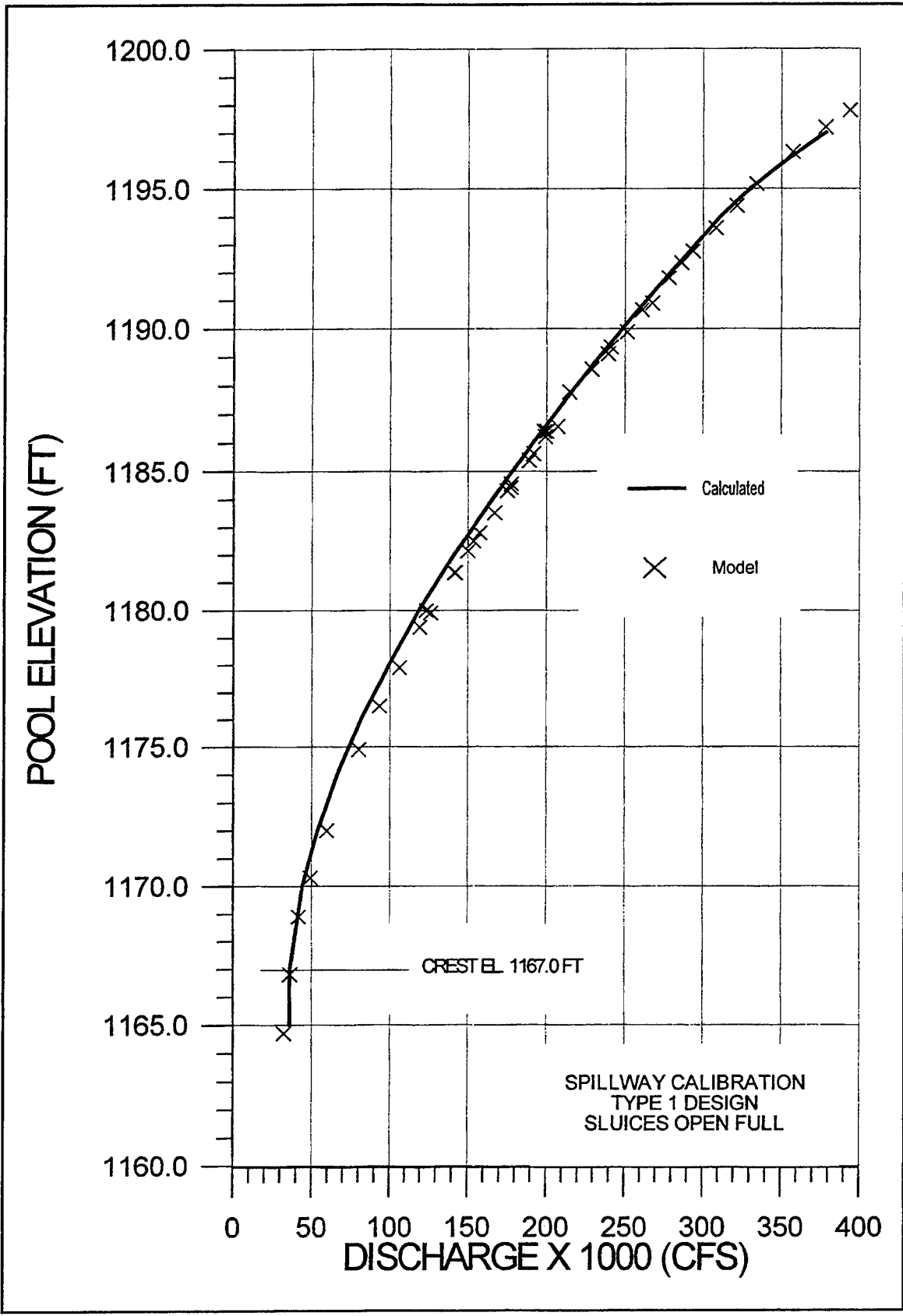
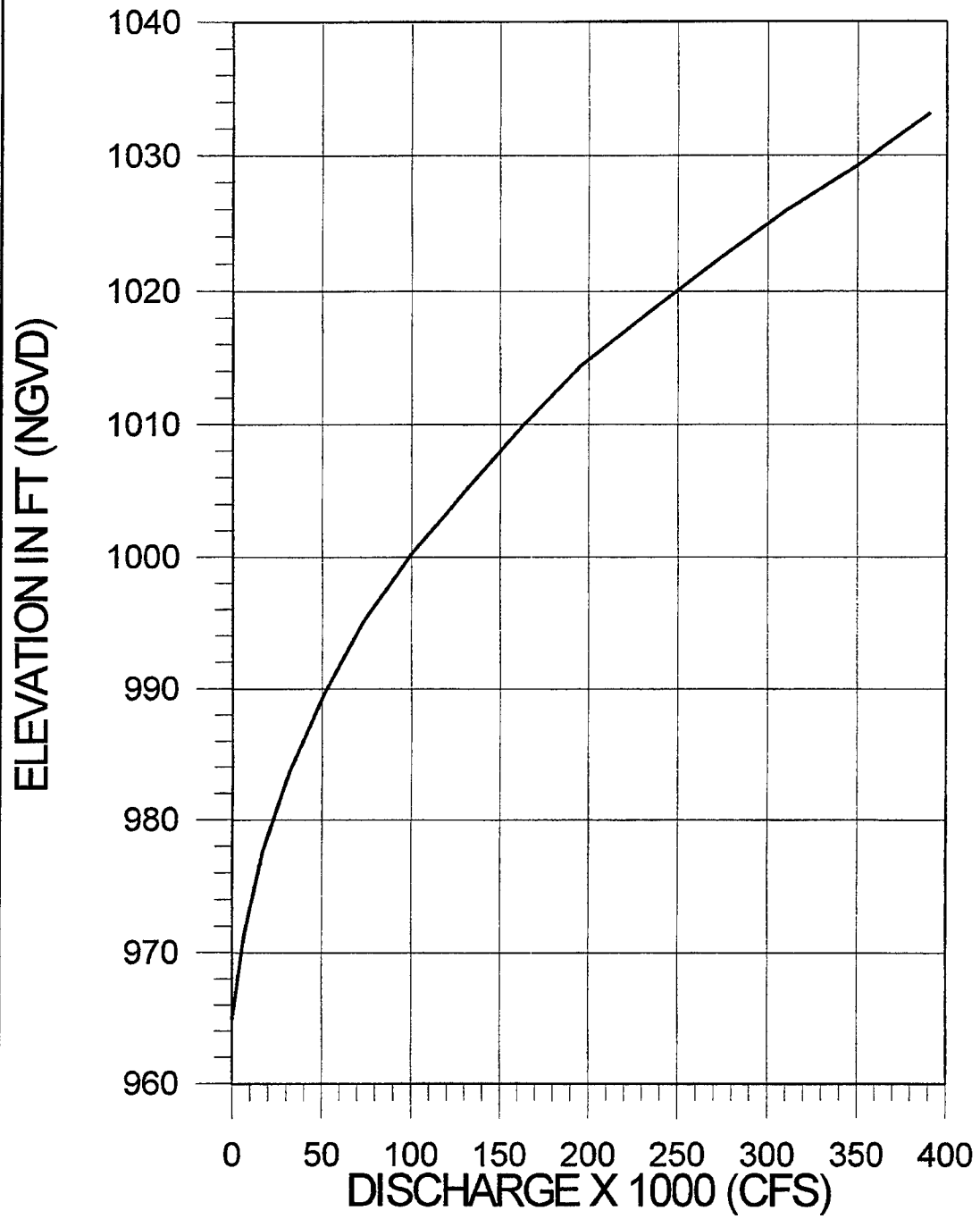


Plate 4



TYGART DAM  
STAGE - DISCHARGE CURVE  
1100 FT D/S OF STILLING WEIR

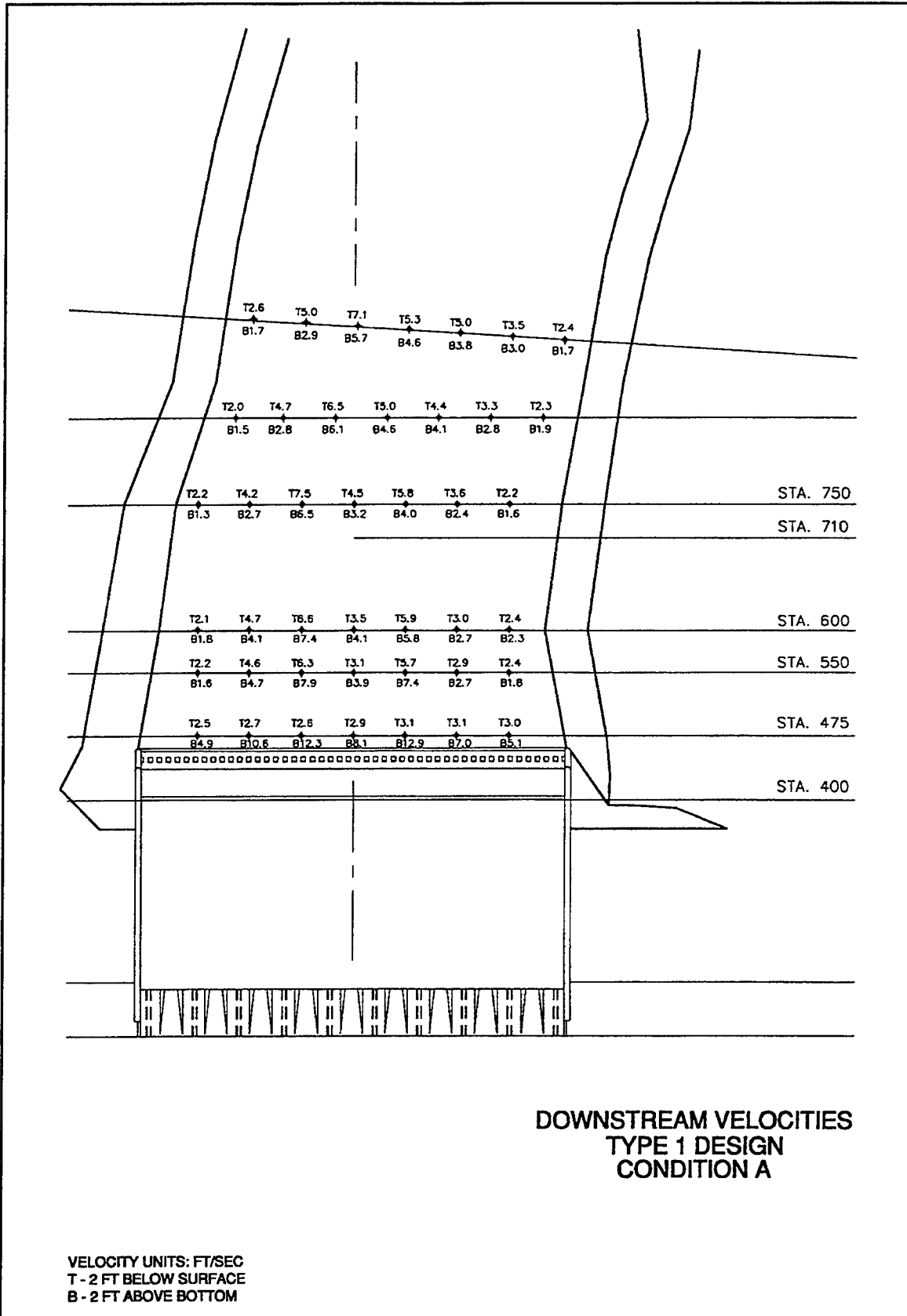
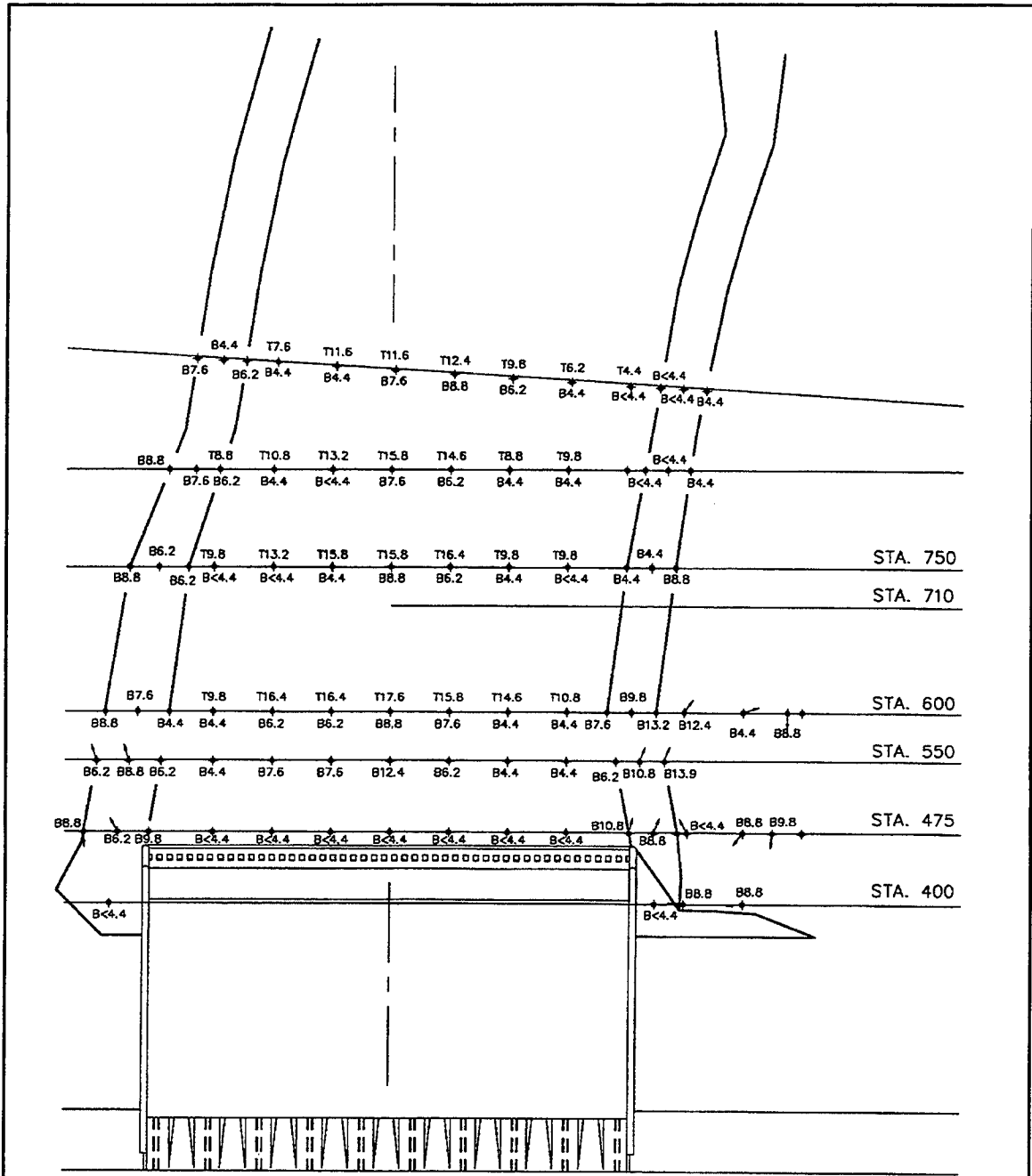
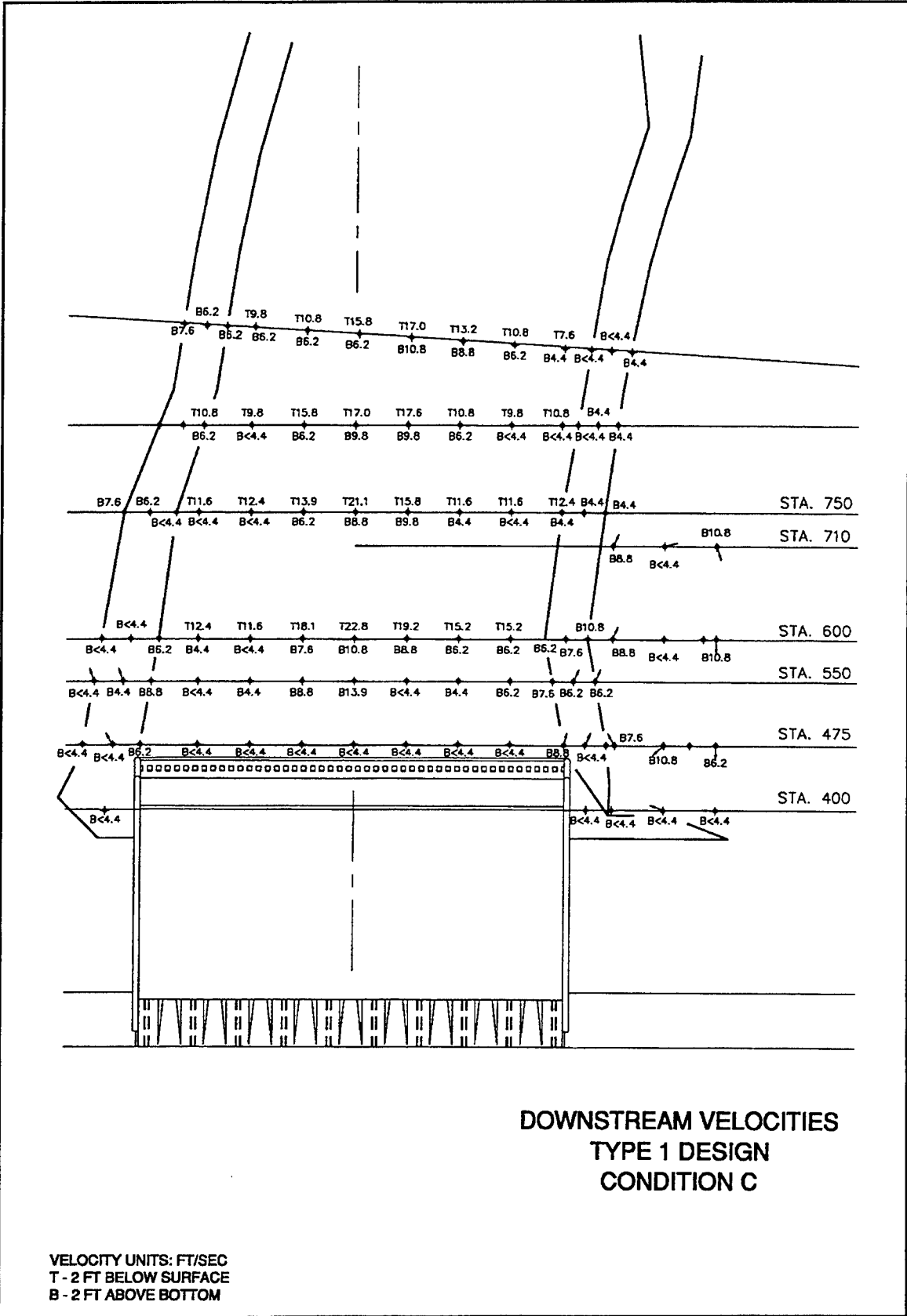


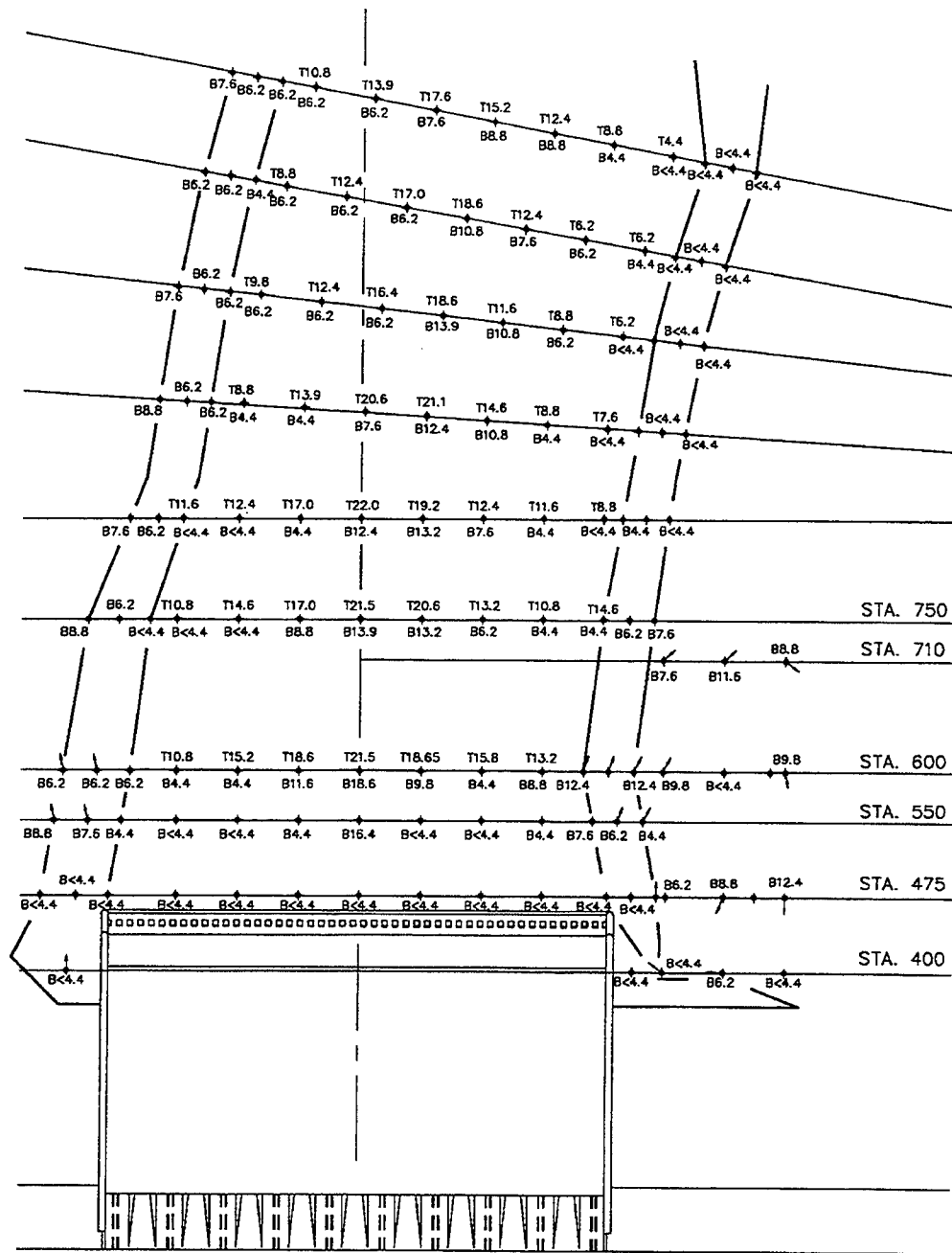
Plate 6



**DOWNSTREAM VELOCITIES  
TYPE 1 DESIGN  
CONDITION B**

VELOCITY UNITS: FT/SEC  
T - 2 FT BELOW SURFACE  
B - 2 FT ABOVE BOTTOM





**DOWNSTREAM VELOCITIES  
TYPE 1 DESIGN  
CONDITION D**

VELOCITY UNITS: FT/SEC  
T - 2 FT BELOW SURFACE  
B - 2 FT ABOVE BOTTOM

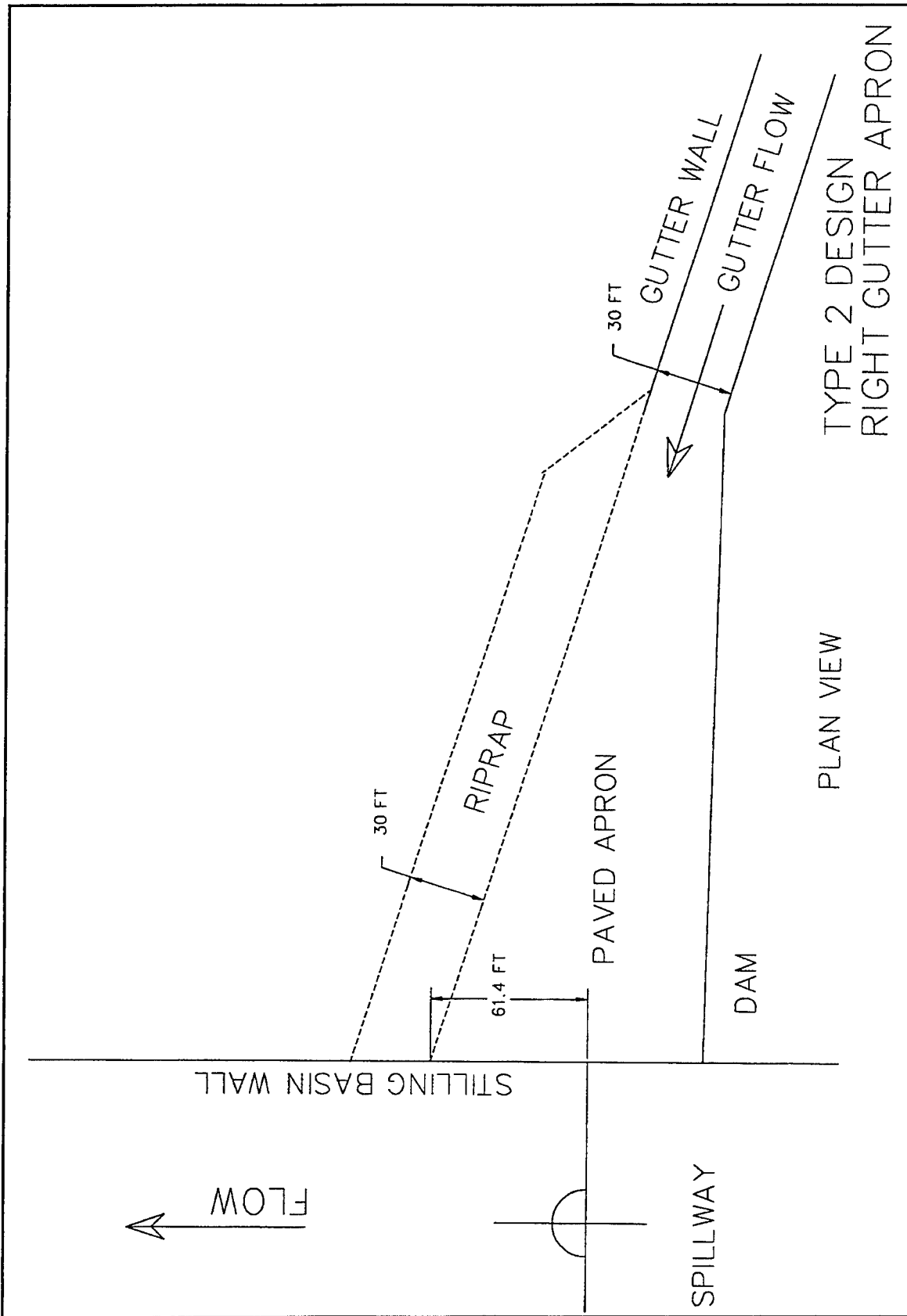
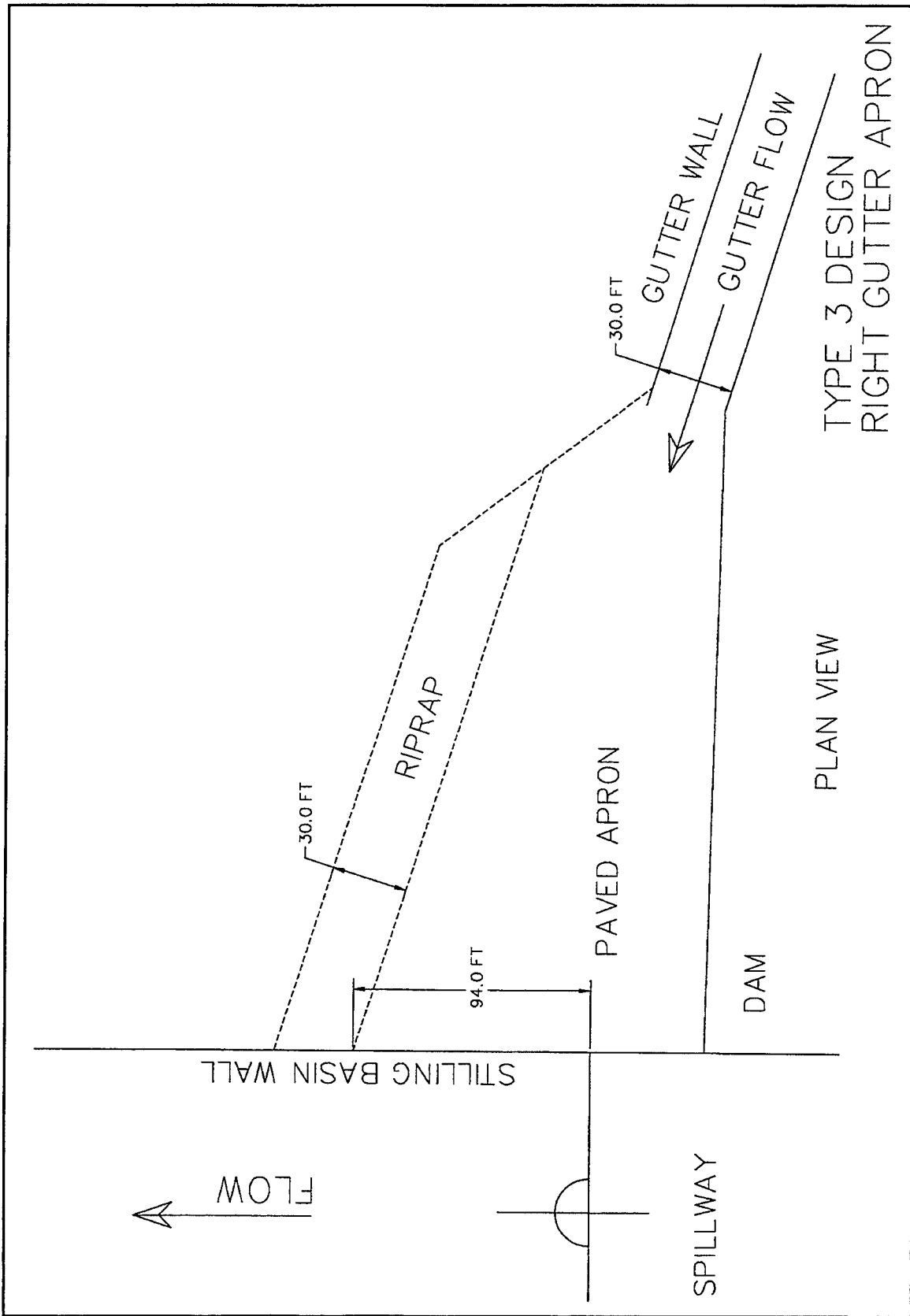
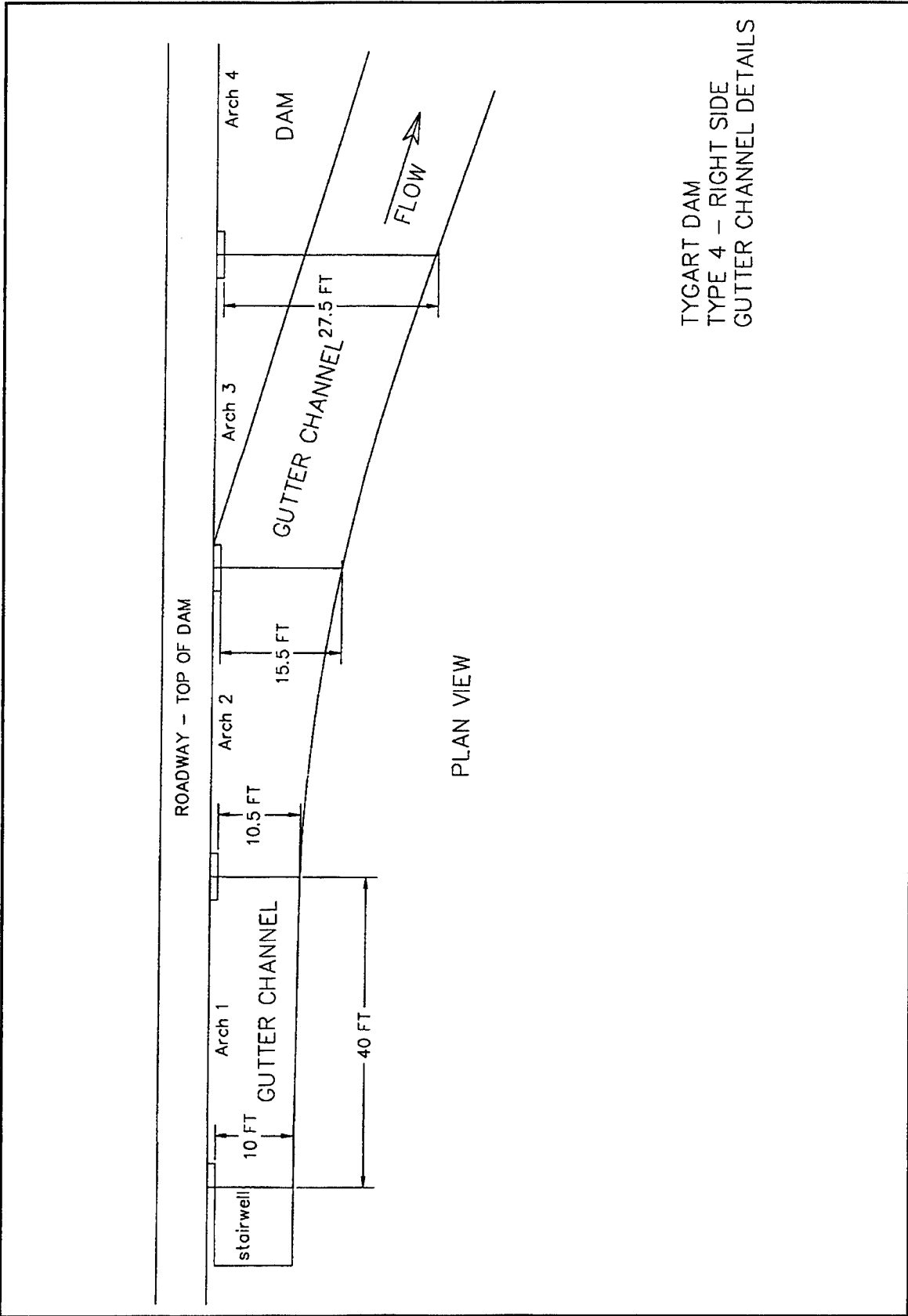
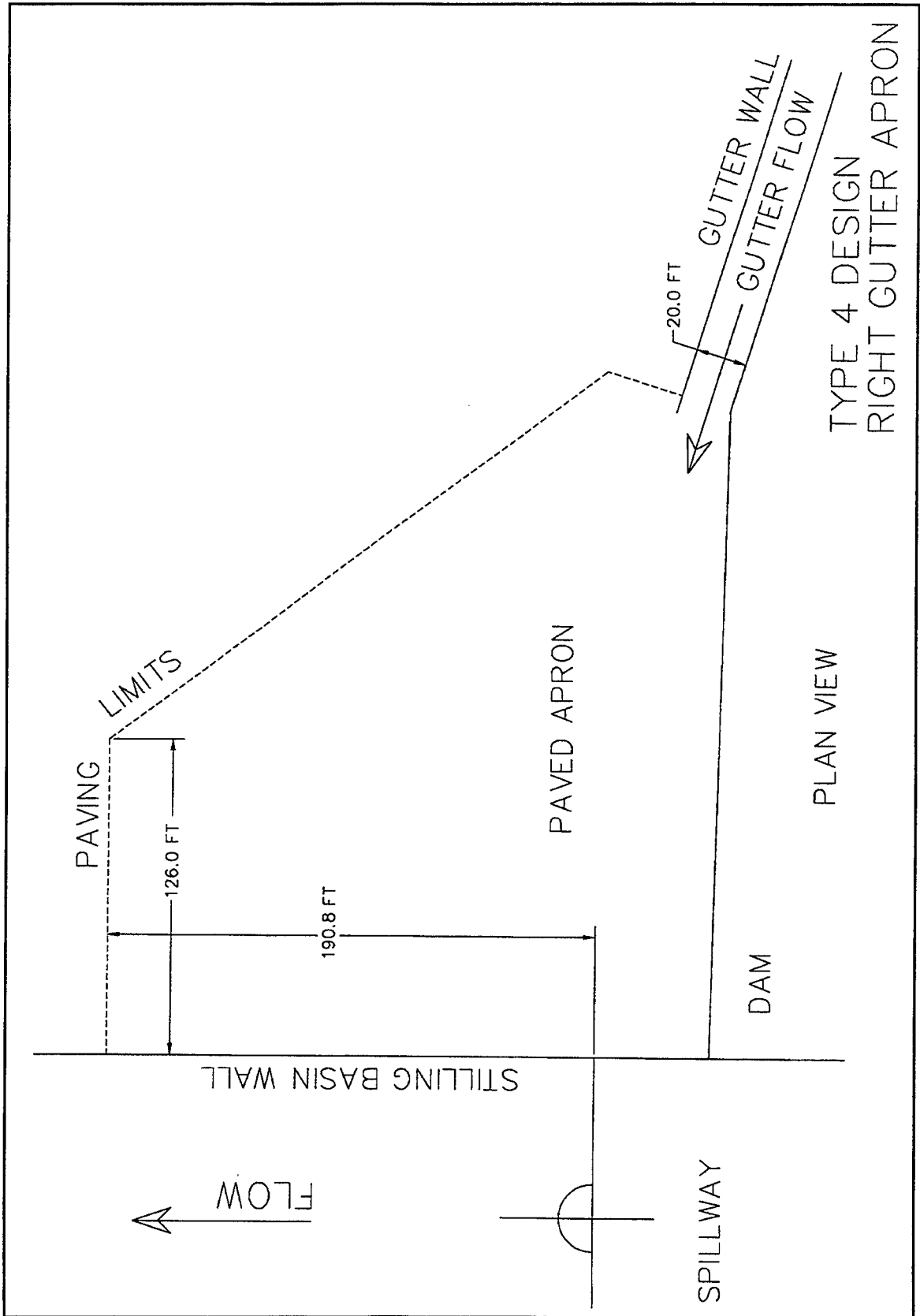


Plate 10



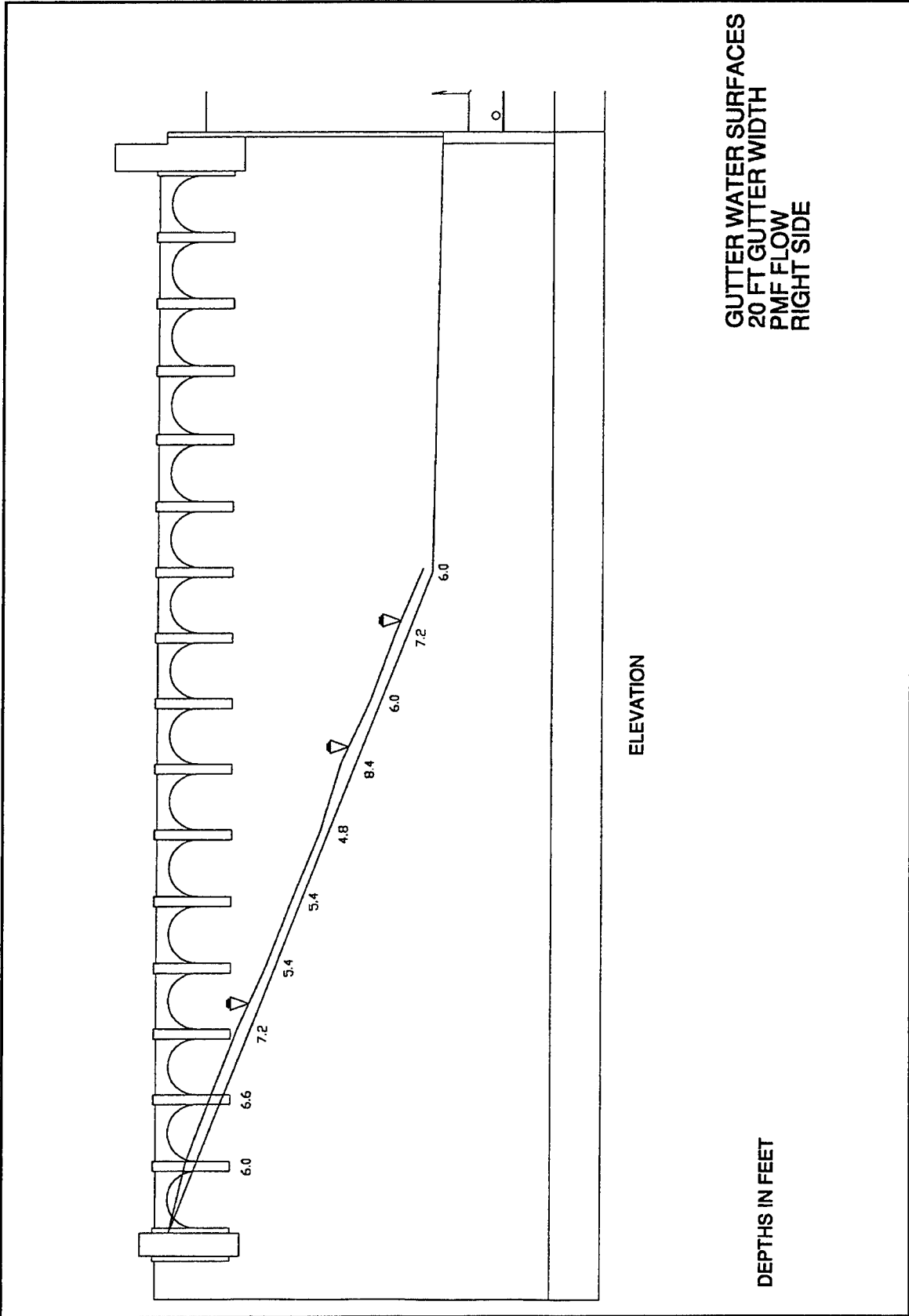


TYGART DAM  
 TYPE 4 - RIGHT SIDE  
 GUTTER CHANNEL DETAILS



TYPE 4 DESIGN  
 RIGHT GUTTER APRON

PLAN VIEW

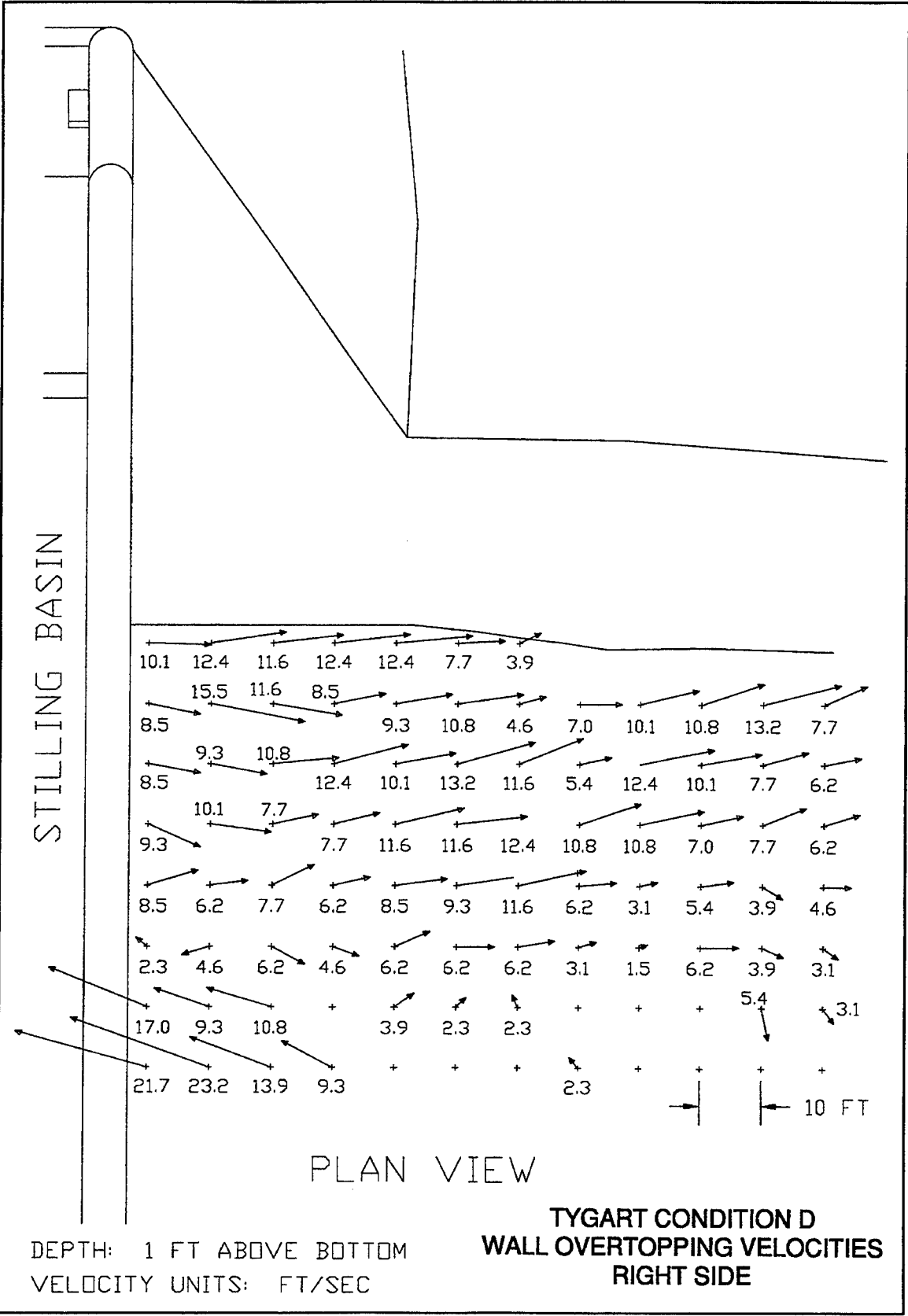


GUTTER WATER SURFACES  
 20 FT GUTTER WIDTH  
 PMF FLOW  
 RIGHT SIDE

ELEVATION

DEPTHS IN FEET

Plate 14



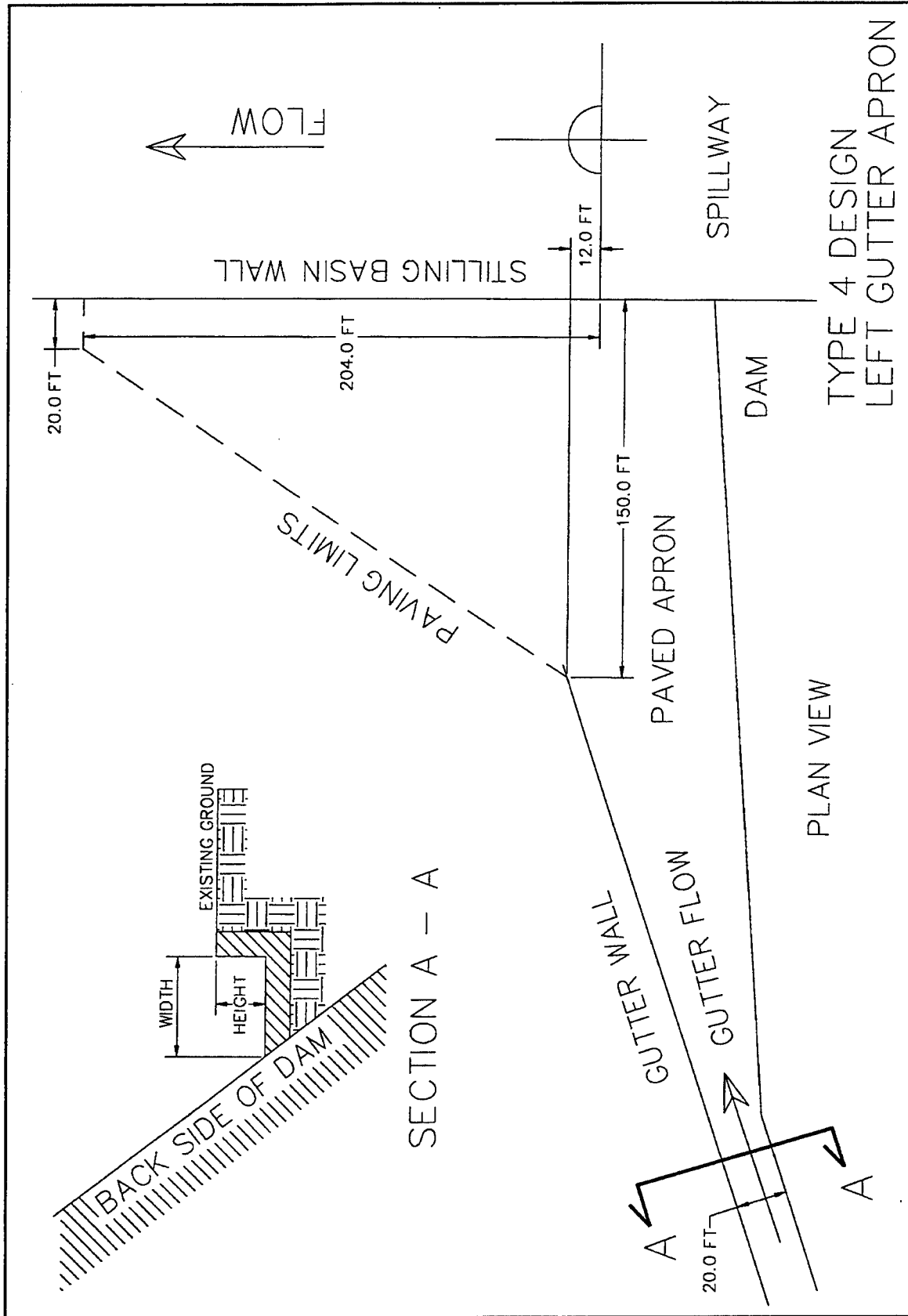
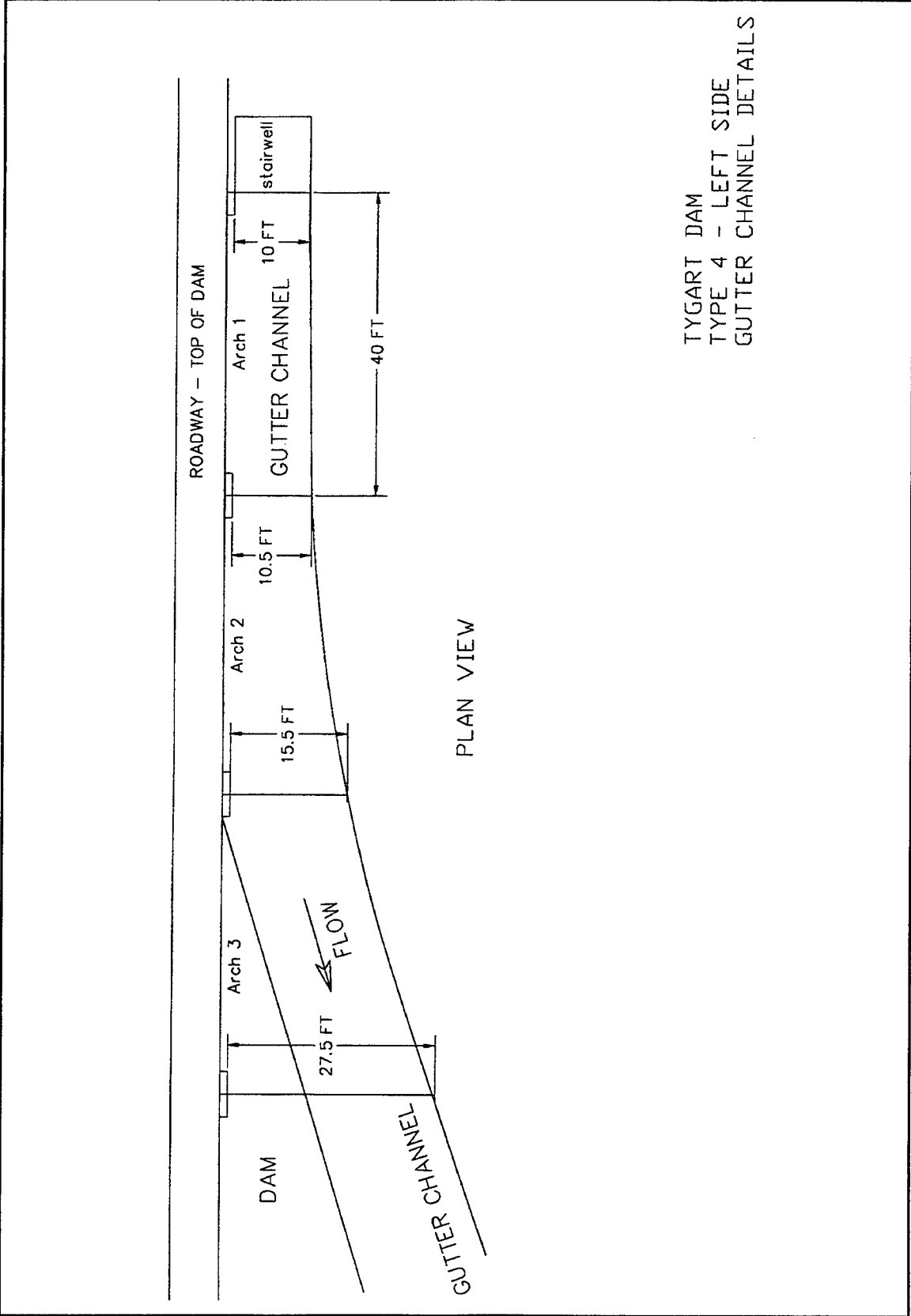
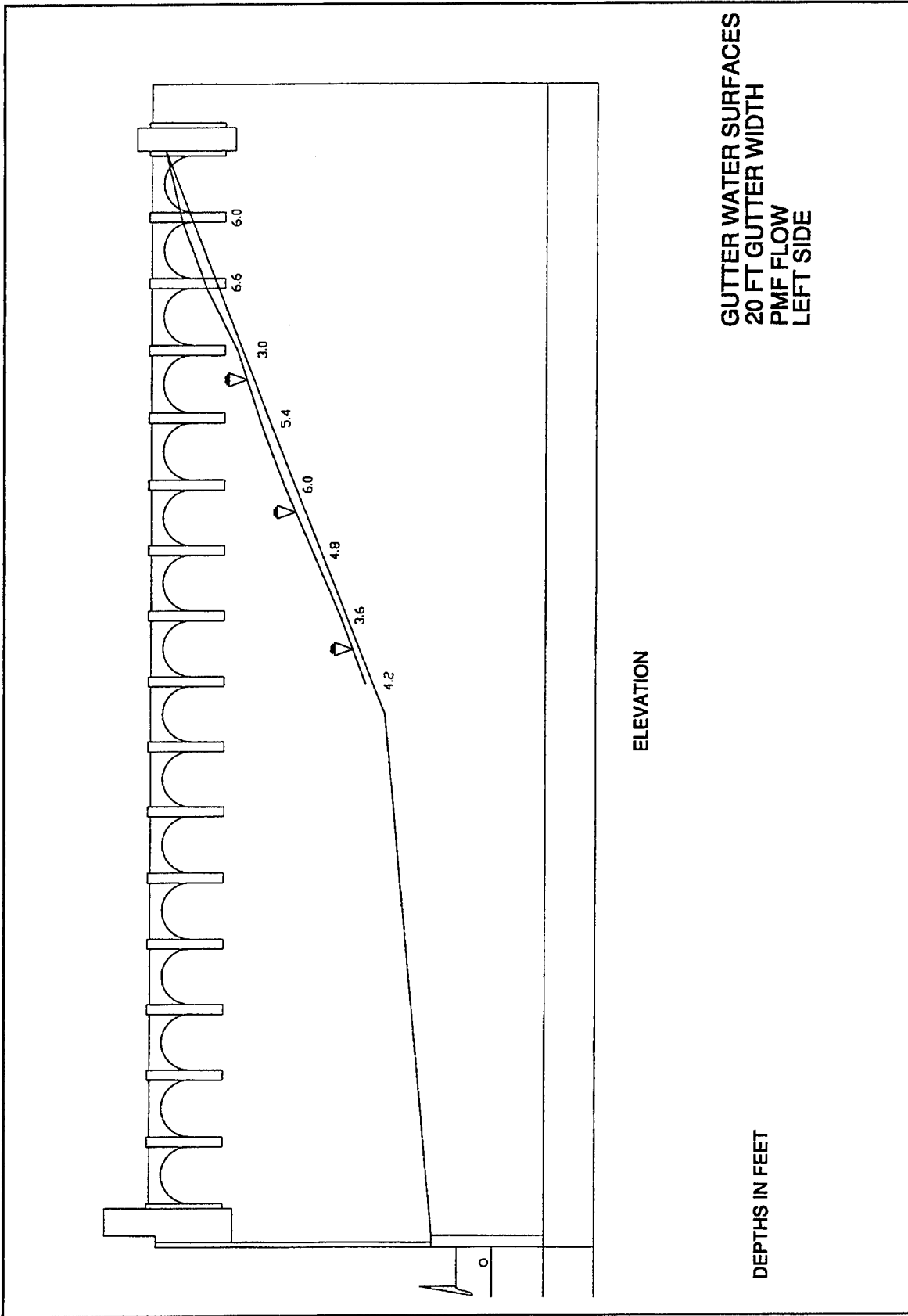


Plate 16



TYGART DAM  
 TYPE 4 - LEFT SIDE  
 GUTTER CHANNEL DETAILS

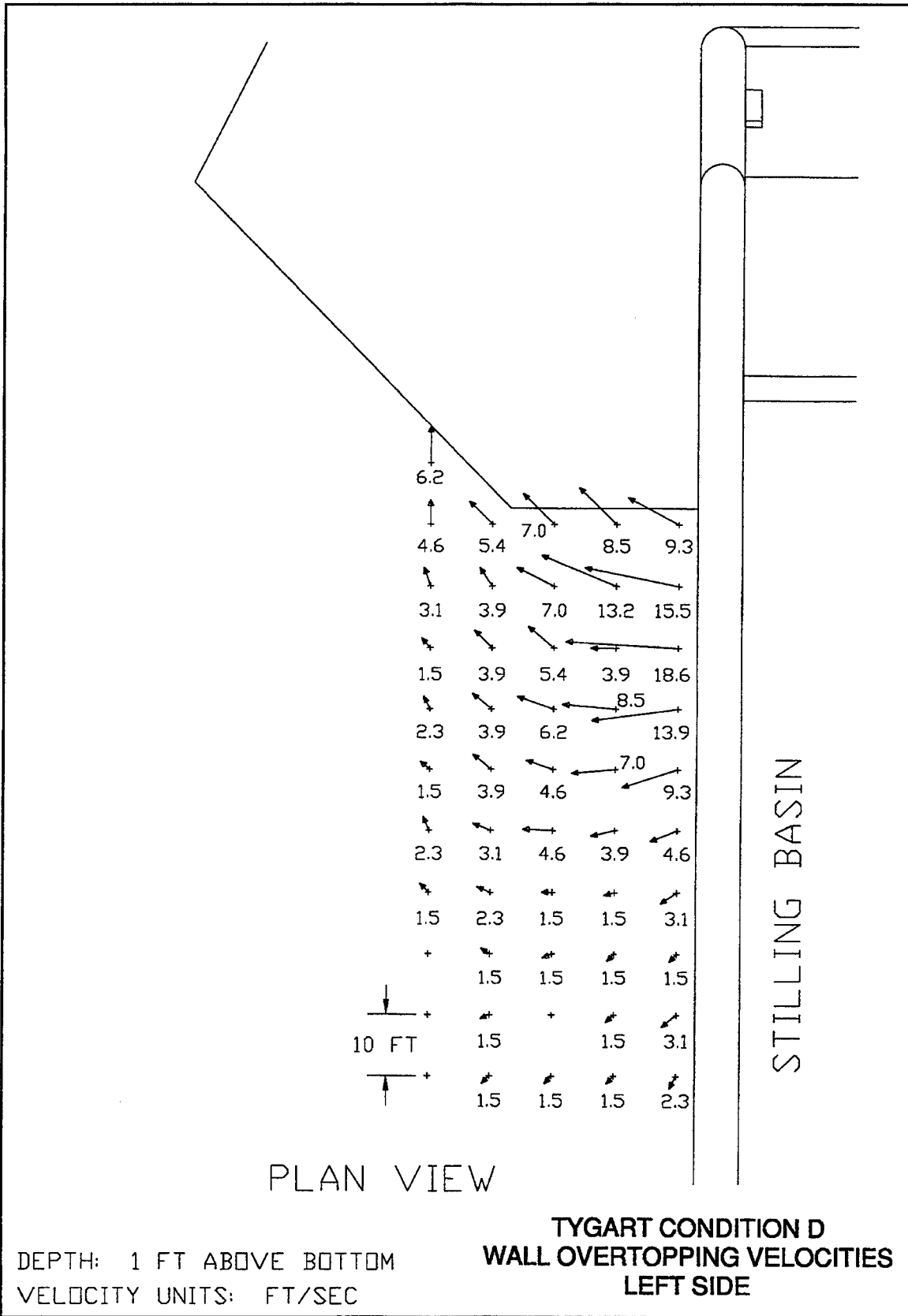


ELEVATION

GUTTER WATER SURFACES  
 20 FT GUTTER WIDTH  
 PMF FLOW  
 LEFT SIDE

DEPTHS IN FEET

Plate 18





# REPORT DOCUMENTATION PAGE

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<b>6. AUTHOR(S)</b> Herman O. Turner, Jr.	
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<b>7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)</b> U.S. Army Engineer Waterways Experiment Station 3909 Halls Ferry Road Vicksburg, MS 39180-6199	<b>8. PERFORMING ORGANIZATION REPORT NUMBER</b> Technical Report CHL-99-7
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<b>13. ABSTRACT (Maximum 200 words)</b>  The Tygart Dam probable maximum precipitation (PMF) was reviewed by a Dam Safety Assurance Study and found to be hydraulically deficient. During the PMF, overtopping flows will flow down the back side of the dam and scour foundation material from the toe of the dam. The resulting loss of stability will cause failure of the dam because of the sliding or overturning of one or more monoliths.  Tygart Dam was reproduced in a 1:60-scale physical model to study ways of handling the overtopping flow. The model and testing conditions were reproduced according to prototype data received from the U.S. Army Engineer District, Pittsburgh. Experiments conducted on the structure showed that the overtopping flow can be routed into the stilling basin and prevent scour. This is accomplished by constructing gutter channels on the back side of the dam at ground level. Experiments also showed that the existing stilling basin and downstream channel can adequately handle the increased discharge.
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<b>14. SUBJECT TERMS</b> Dam failure Dam Safety Assurance Study Gutter channel Hydraulic models	Overtopping flow Tygart Dam Tygart River		<b>15. NUMBER OF PAGES</b> 65
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