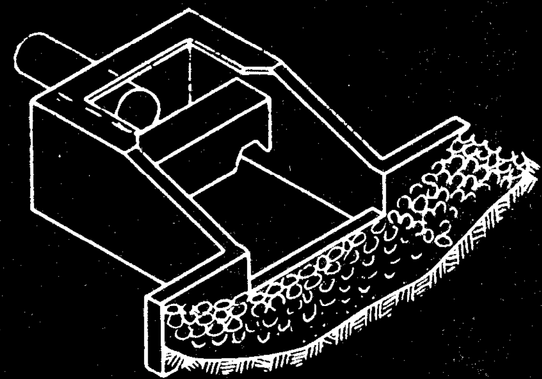


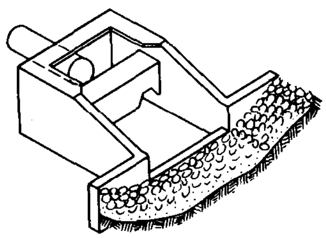
Hydraulic Design of Stilling Basin for Pipe or Channel Outlets

A Water Resources Technical Publication

RESEARCH REPORT NO. 24

United States Department of the
INTERIOR
Bureau of Reclamation

Research Report No. 24 • A WATER RESOURCES TECHNICAL PUBLICATION



Hydraulic Design of Stilling Basin for Pipe or Channel Outlets

Basin VI in the Bureau of Reclamation designation

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UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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PREFACE

High-energy forces in flowing or falling water must be contained or dissipated to prevent damaging scour or erosion of downstream channels.

Various means for energy dissipation are employed at hydraulic installations. Stilling basins are among the most common. Ten types, I through X, are used by the Bureau of Reclamation. (The Roman numeral classifications are internal Bureau designations.) The variety of operating conditions necessitates this wide range of stilling basin designs.

Criteria for design of the 10 stilling basin types were first summarized in Engineering Monograph No. 25, published in 1958 and revised in 1963. The monograph was based on a series of earlier papers and laboratory reports.

This study of the type VI stilling basin, which is used for pipe or open channel outlets, was made to standardize and modify existing and previously used procedures in the design of this impact stilling basin.

Development of the type VI short impact-type basin originated with a need for some 50 or more stilling structures on a single irrigation project. Relatively small basins providing energy dissipation independent of a tailwater curve or tailwater of any kind were required.

The information in this report is intended for water resource centers, government agencies, municipal and industrial water operators, and hydraulics and irrigation systems designers.

Included in this publication is an informative abstract with a list of descriptors, or keywords, and identifiers. The abstract was prepared as part of the Bureau of Reclamation's program of indexing and retrieving the literature of water resources development. The descriptors were selected from the *Thesaurus of Descriptors,* which is the Bureau's standard for listing of keywords.

CONTENTS

TABLE

FIGURES

CONTENTS–Continued

INTRODUCTION

This study was conducted to standardize and modify existing procedures used in the design of the impact, type VI stilling basin.

Stilling Basin VI, as referred to in section 6 of report No. Hyd-399 [*1*]* and in Engineering Monograph No. 25 [*2*], is an impact-type energy dissipator, contained in a relatively small boxlike structure which requires no tailwater for successful performance. Although the emphasis in this discussion is placed on use with pipe outlets, the structure may be used with an open channel chute.

It was originally developed for use as an energy dissipator at several locations on the Franklin Canal. Many of these basins are in use on other Bureau projects and generally have been designed in accordance with the procedures outlined in these two publications. However, operation of the various prototype structures has revealed the need for revision of these design standards. Unforeseen operating conditions in the field and the over-generalization of the present design rules have caused operating problems at some of the prototype structures.

Four principal operating problems that have occurred at various installations are: (1) the basin tends to clog with debris upstream of the hanging baffle. Russian thistles and similar weeds are the main source of the debris, which is not generally a problem in cultivated areas. (2) Excessive splash overtopping the compartment walls upstream of the baffle, usually resulting from too small a basin for the quantity and velocity of flow involved, has eroded the fill outside the basin walls. (3) The discharge from the entrance pipe passes under the baffle, resulting in very little energy dissipation in the basin and excessive erosion of the downstream channel. This has occurred with a sloping entrance pipe or with an oversized basin having a horizontal entrance pipe discharging at less than the design flow. (4) Channel erosion at the end of the basin where the size of riprap was not adequate.

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* *Italicized* numbers in brackets refer to references cited at the end of this report.

THE MODELS

Two model basins, 1.6 and 2.4 feet (490 and 730 millimeters) wide, were constructed. The other dimensions were related to the width of the basin, as shown in figure 1.

An 8-inch (200-mm) inside-diameter pipe was used at the entrance to each of the basins. Deflectors of various sizes were installed on the crown of this pipe upstream of the portal to vary the velocity of flow entering the basins. One-fourth, one-half, three-fourths, and full pipe flows were used in the tests, as well as flow from an 8-inch (200-mm) rectangular open channel.

Each of the two basins discharged into a canal section lined with 1.5-inch (38-mm) gravel. Tailwater elevations were controlled by stoplogs at the end of the canal section. The bottoms of the canal sections were at the same elevation of the basin end sill and were as wide and as long as the basin width. The side slopes were 1.5 to 1 for the 2.4-foot (730-mm) wide basin and vertical for the smaller basin.

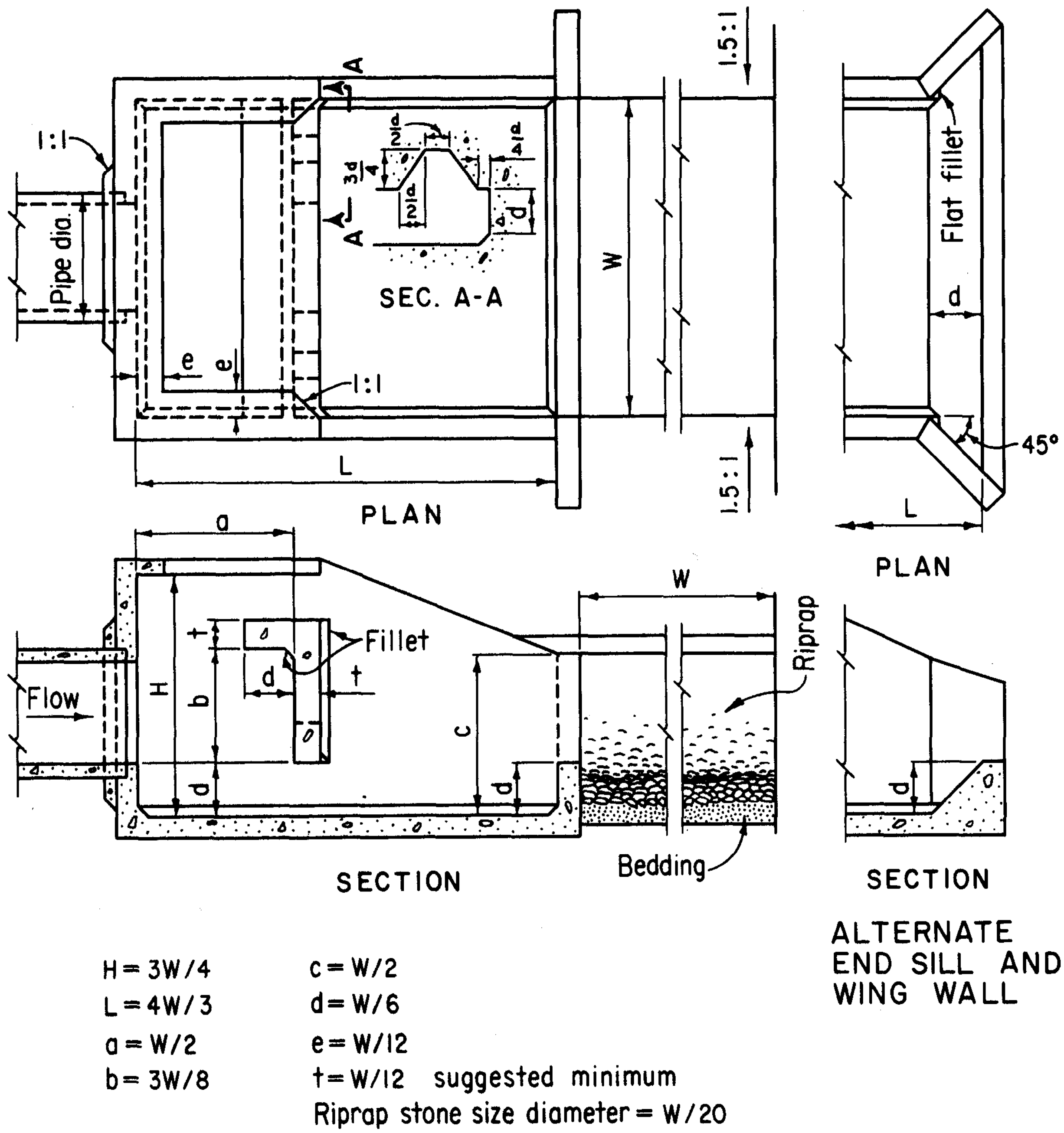

Figure 1.–General design of the type VI impact stilling basin.

THE INVESTIGATION

The investigation was primarily concerned with relating the basin size to the discharge and velocity and in relating the downstream channel and riprap requirements to the basin size. It was also concerned with special situations involving debris, silt, tailwater, sloping entrance pipe, and rectangular open channel entrances not usually encountered in the standard design of the type VI basin.

Standardization of the Basin Dimensions in Terms of Basin Width

A test basin was constructed having dimensions related to the basin width in accordance with those developed for the basin in the earlier study [*2*].

To test the adequacy of this 2.4-foot (730-mm) wide model basin, tests were conducted over a range of flows that had been determined in the earlier tests [*2*] to be the limits of exceptionally mild operation and of safe maximum operation for a given basin width, provided the entrance flow velocity did not exceed 30 feet per second (9.1 m/s) (prototype).

These test discharges were related to the basin width in accordance with the equation:

$$Q = (W/C)^{2.5}$$

where "Q" is the discharge in cubic feet per second, "W" is the inside width of basin in feet, and "C" is a coefficient that varies for the maximum, minimum, and intermediate flows. The coefficient in English units is 1.46 for the maximum permissible flow, 1.80 for the minimum mild flow, and 1.60 for the intermediate flow.[1] Each test discharge was run at approximately half full and full pipe to obtain high- and low-entrance velocity conditions and with controlled and uncontrolled tailwater depths (figs. 2 and 3).

The larger flows with the higher velocities intermittently surged and splashed high on the basin walls immediately downstream from the hanging baffle and overtopped the sidewalls at the downstream end of the basin (figs. 2 and 3). To improve these flow conditions, the width of the notches in the baffle was reduced and the notches moved a short distance away from the sidewalls. Also, the slope of the top of the basin sidewalls was reduced to increase the height of the wall at the downstream end of the basin. The modification to the notches reduced the splashing and the height of the water surface rise on the sidewalls. Increasing the height of the sidewalls provided additional freeboard at the downstream end of the basin. These modifications are incorporated into the standard design dimensions shown in figure 1.

Standardization of the Basin Flow Entrance

The flow will usually enter the basin from a circular pipe but may enter from a rectangular open channel. The pipe may flow full or partially full. If it flows partially full and the upstream entrance to the pipe is submerged, the pipe should be vented to the atmosphere. The vent should be located near the upstream end of the pipe and have a diameter of about one-sixth the pipe diameter.

Although the entrance pipe or channel is usually horizontal or on a very slight downward grade, some installations may require an entrance pipe on a relatively steep slope. The hydraulic performance of the 2.4-foot (730-mm) wide model basin was determined with the entrance pipe sloped downward about 12°. Both high- and low-velocity test flows partially impinged on the hanging baffle and the bottom of the baffle was only partially submerged, resulting in incomplete energy dissipation.

The model tests showed that a horizontal fillet on the invert of the pipe for a distance of one pipe diameter upstream from the portal caused greater jet impingement on the baffle, deeper submergence of the bottom of the baffle, and consequently better energy dissipation. The same improvement could be obtained by placing the entrance pipe horizontally for a distance of one or more pipe diameters upstream from the basin entrance. Either of these two methods may

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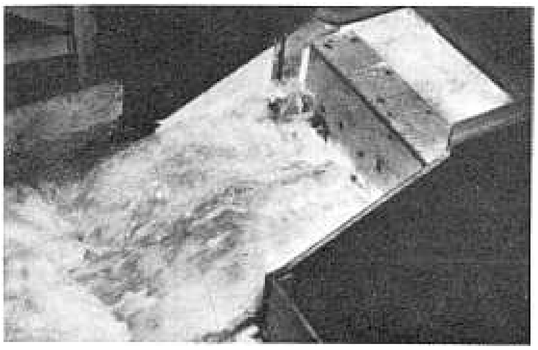
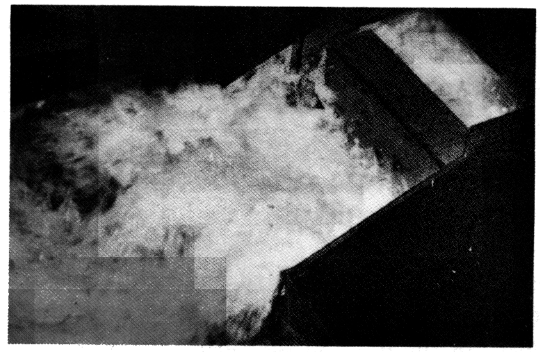
[1] 1 ft^3/s = 28.3 L/s = 28.3 x 10^{-3} m^3/s. To obtain the discharge in cubic meters per second (m^3/s), the width must be in meters and the discharge coefficient must be multiplied by 1.27. To obtain the discharge in liters per second (L/s), either multiply the cubic meters per second value by 1000, or calculate with the width in millimeters and the discharge coefficient multiplied by 80.

be used for entrance pipe slopes up to 15°. Entrance pipes having a downward grade exceeding 15° should be horizontal for at least two diameters upstream from the basin entrance.

Replacing the sloping entrance pipe in the model with an 8-inch (200-mm) wide rectangular channel on a similar slope did not change the hydraulic performance of the basin. However, flow from the basin backed up into the open channel, making it necessary to raise the channel walls to the same height as the basin walls. To further contain the flow, the invert of the channel should be horizontal for a distance equivalent to at least two channel widths upstream from the basin entrance.

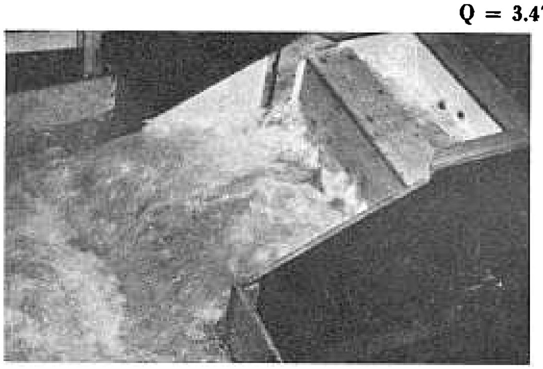


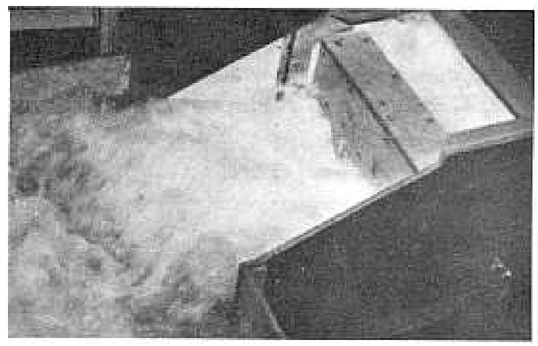


V = 9.9 ft/s (3.0 m/s) V = 21.58 ft/s (6.6 m/s)

C = 1.46

Q = 3.47 ft^3/s (98 L/s)

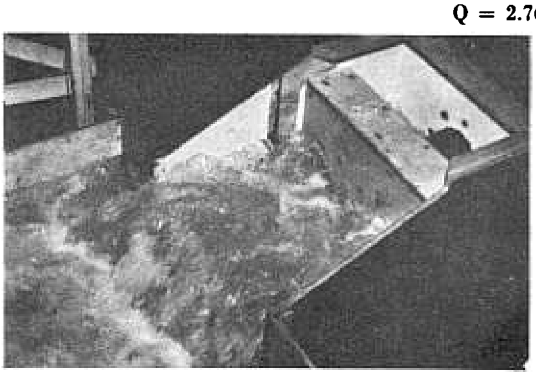


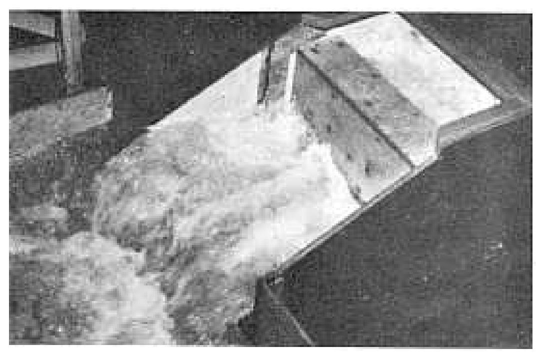


V = 7.89 ft/s (2.4 m/s) V = 17.15 ft/s (5.2 m/s)

C = 1.60

Q = 2.76 ft^3/s (78 L/s)





V = 5.84 ft/s (1.8 m/s) V = 12.69 ft/s (3.9 m/s)

C = 1.80

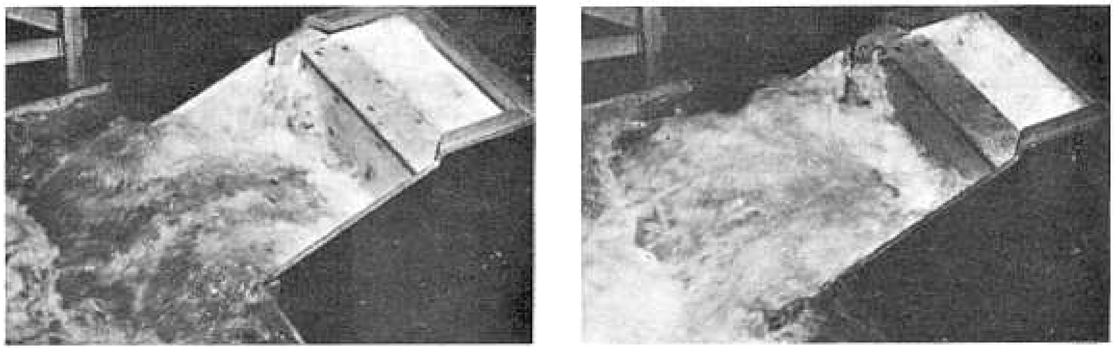
Q = 2.04 ft^3/s (58 L/s)

Q = $(W/C)^{2.5}$ where W=basin width of 2.4 feet (730 mm)

V = velocity of flow at entrance

Tailwater elevation in tailbox is below basin end sill

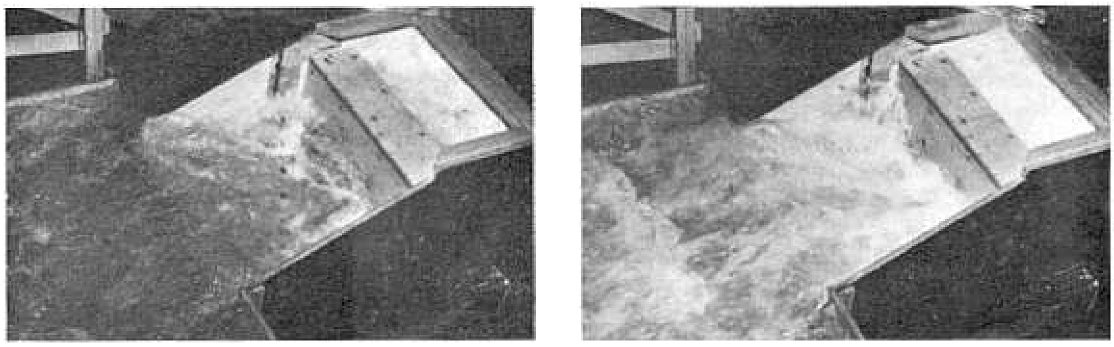
Figure 2.–Test flows with uncontrolled tailwater.



V = 9.9 ft/s (3.0 m/s) V = 21.58 ft/s (6.6 m/s)

C = 1.46

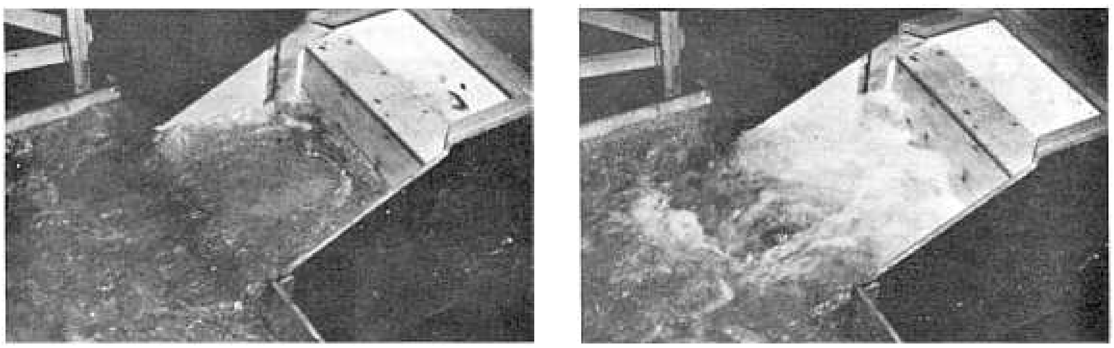
Q = 3.47 ft³/s (98 L/s)



V = 7.89 ft/s (2.4 m/s) V = 17.15 ft/s (5.2 m/s)

C = 1.60

Q = 2.76 ft³/s (78 L/s)



V = 5.84 ft/s (1.8 m/s) V = 12.69 ft/s (3.9 m/s)

C = 1.80

Q = 2.04 ft³/s (58 L/s)

$Q = (W/C)^{2.5}$ where W = basin width of 2.4 feet (730 mm)

V = velocity of flow at entrance

Tailwater elevation in the tailbox is at d + b/2 (see fig. 1)

Figure 3.–Test flows with controlled tailwater.

Standardization of the Basin Size

After standardizing the basin dimensions in relation to the basin width, the next step was to standardize the size in relation to the quantity and velocity of the flow entering the basin. The basin size is represented by the basin width, the quantity and velocity of flow by the Froude number of the incoming jet.

It was believed that the shape of the incoming jet was relatively unimportant in evaluating the adequacy of a type VI basin. Therefore, to standardize the method of computing the Froude number of the incoming flow, it was assumed that the cross sectional area of the jet in the circular pipe or rectangular channel had the shape of a square; thus, the depth of the incoming flow "D" was considered to be the square root of its cross sectional area.

The test flows (figs. 2 and 3) used in verifying the standard dimensions of the basin in reference II were repeated in the 2.4-foot (730-mm) wide model basin, but with a riprapped channel simulated at the downstream end of the basin. Water surface roughness and erosion, together with the ability of the basin to contain the flow, were used as guidelines in evaluating the hydraulic performance test flows (figs. 4 through 7). Each of the test flows was judged to be satisfactory or unsatisfactory and plotted in dimensionless terms (Froude number of the incoming flow "F" versus the ratio of basin width to the incoming depth of the flow "W/D") in figure 8.

To increase the range of data to be evaluated for figure 8, the cross sectional area of the incoming flow was reduced to one-fourth the area of the 8-inch (200-mm) pipe, and the velocity of the flow entering the 2.4-foot (730-mm) wide model basin was increased (fig. 9). Thus, both the Froude number and the width/depth ratio increased. The width/depth ratio for these tests was 8.15, at which the Froude number of a theoretical square jet at the entrance was 6.70 for the minimum satisfactory operation. Because the size of the jet was becoming very small in relation to the width of the basin, the design curve in figure 8 was not extended beyond a width to depth ratio of 10, which corresponded to flow having a Froude number of about 9.

To increase the range of data in the other direction, the cross sectional area of the incoming flow was increased in relation to the basin width by switching to the 1.6-foot (490-mm) wide model while maintaining the 8-inch (200-mm) entrance pipe (fig. 10). The tests were evaluated and plotted in figure 8 at a W/D ratio of 3.08. The side slopes of the downstream discharge channel were vertical and the same distance apart as the basin sidewalls. Although this was not typical of the usual prototype installation and is not recommended, it was not considered to be critical in evaluating the performance of the basin. For these tests, the Froude number was in the vicinity of 1.0 and the height of the incoming flow was near the top of the baffle. Therefore, it did not appear practical to design this basin for W/D ratios smaller than 3, which corresponds to a flow having a Froude number of 1.1.

Additional tests were run in this smaller model (figs. 11 and 12) to confirm the findings found in the larger model basin. The results of these tests are plotted in figure 8 at W/D ratios of 3.8 and 6.1. The two models showed very good agreement in what was considered satisfactory and unsatisfactory performance, as seen by comparing figures 4 and 6 with 11 and 12.

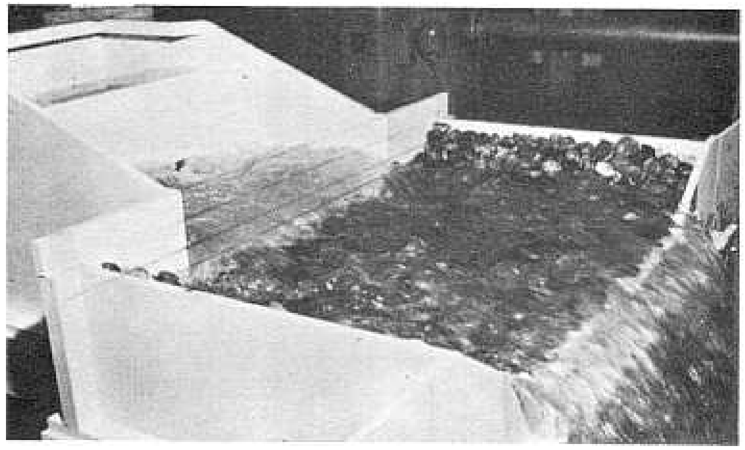
In figure 8, the straight line drawn through the data points with the highest Froude numbers for which satisfactory operation existed indicates the minimum width of basin that can be used for a given Froude number. Data points above the line indicate that it should be permissible to increase the size of the basin approximately 25 percent; however, this should not be done as these points represent the condition when the basin is operating at less than the design discharge. If the basin is too large, the incoming jet will pass under the baffle as has occurred at some installations and effective energy dissipation will not occur. For best results, the basin should be designed for the minimum width indicated in figure 8.

Standardization of the Entrance Velocity Limitation

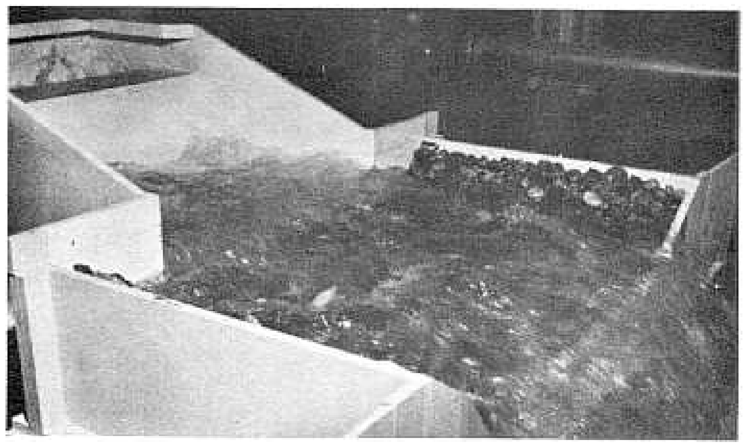
In previous studies [*2*], the design criteria for this type of structure were based on discharge alone. The maximum incoming velocity was arbitrarily limited to 30 feet per second (9.1 m/s). However, some prototype structures have been designed and operated at velocities exceeding this limit. The type VI stilling basins for the outlet works of Picacho South and North Dams were designed for velocities up to 39 and 48 feet per second (11.9 and 14.6 m/s), respectively, for flows of 165 and 275 cubic feet per second (4.7 and 7.8 m^3/s), respectively. They have operated satisfactorily at 80 percent capacity at velocities of 32 and 37 feet per second (9.8 and 11.3 m/s) (fig. 13).

To prevent the possibility of cavitation or impact damage to the basin, the maximum entrance velocity should be limited to about 50 feet per second (15 m/s). At this velocity the maximum Froude number, 9.00, for which the basin is recommended will occur at a design flow of 46 cubic feet per second (1.3 m^3/s). For Froude numbers less than about 4, this basin would not be feasible at this velocity because of the enormous size of the structure involved.

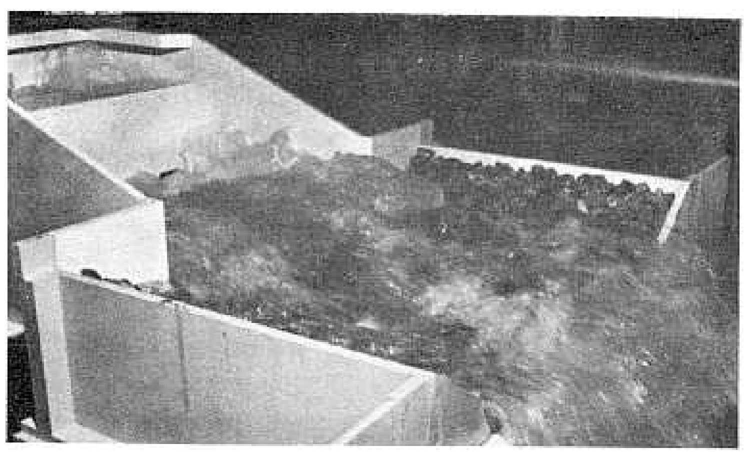
F = 1.34
W/D = 4.06
Satisfactory



F = 1.81
W/D = 4.06
Satisfactory

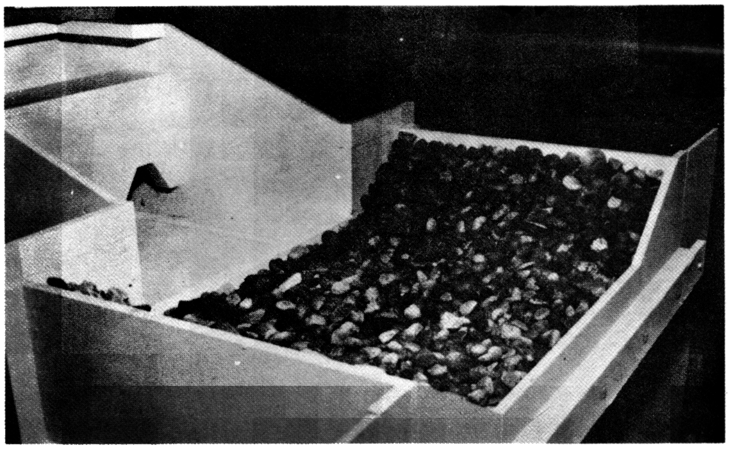


F = 2.27
W/D = 4.06
Unsatisfactory

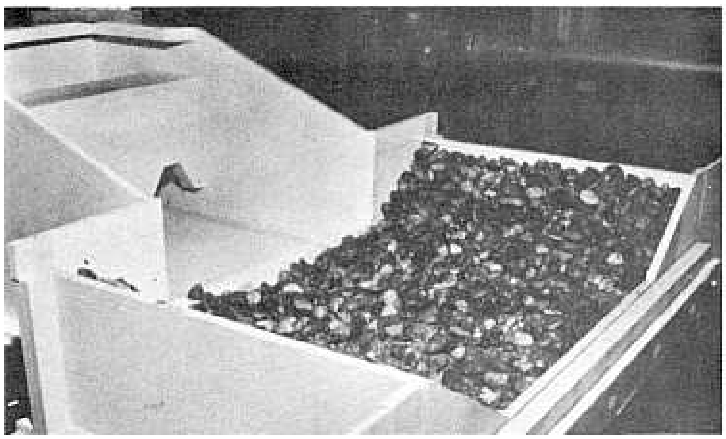


Note: For erosion results, see figure 5; for plot of these operating conditions, see figure 8.

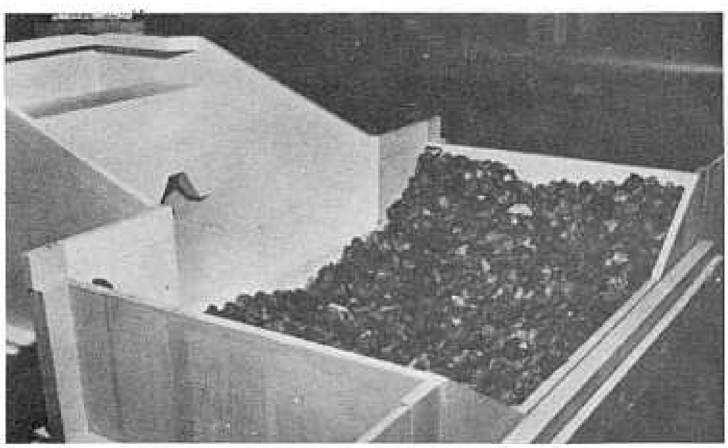
Figure 4.–Entrance pipe flowing full with uncontrolled tailwater in 2.4-foot (730-mm) wide basin.



F = 1.34
W/D = 4.06
No erosion
Satisfactory



F = 1.81
W/D = 4.06
Erosion
Satisfactory

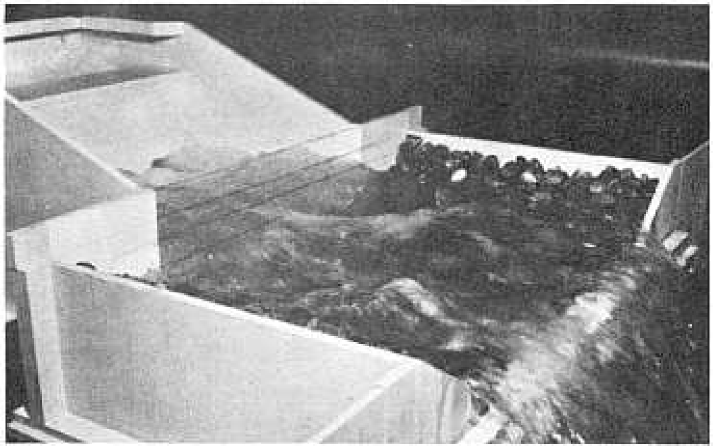


F = 2.27
W/D = 4.06
Excessive erosion
Unsatisfactory

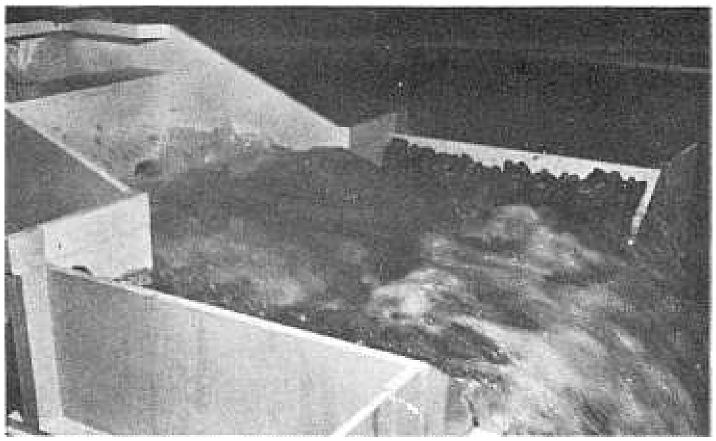
Note: For plot of these operating conditions, see figure 8.

Figure 5.–Erosion for uncontrolled tailwater with entrance pipe flowing full in 2.4-foot (730-mm) wide basin

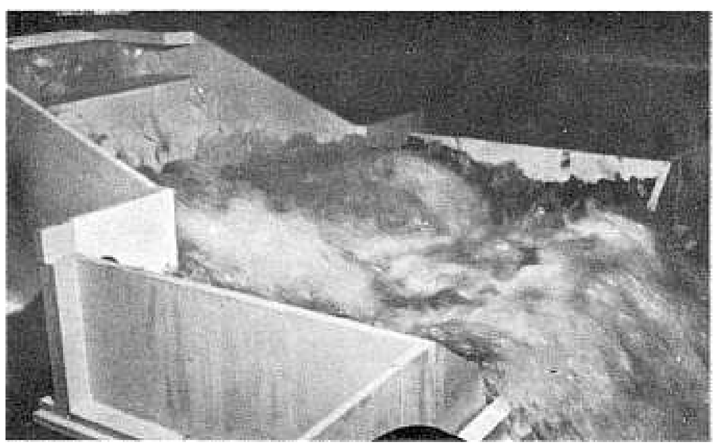
F = 3.53
W/D = 5.98
Satisfactory



F = 4.77
W/D = 5.98
Unsatisfactory

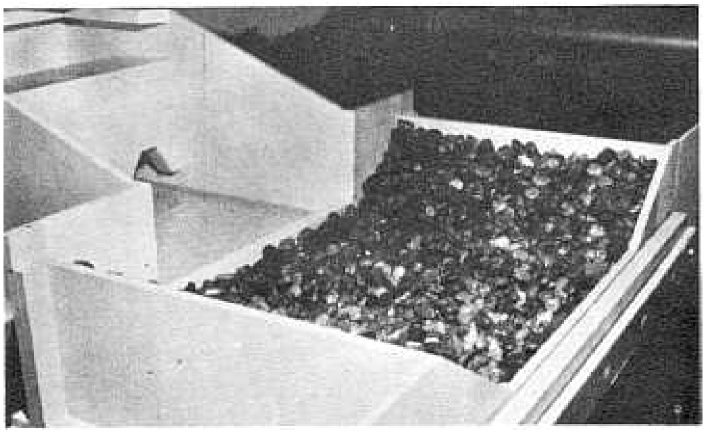


F = 6.01
W/D = 5.98
Unsatisfactory

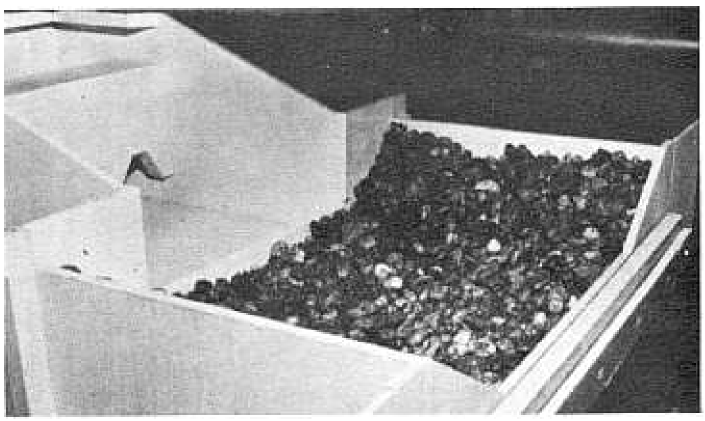


Note: For erosion results, see figure 7; for plot of these operating conditions, see figure 8.

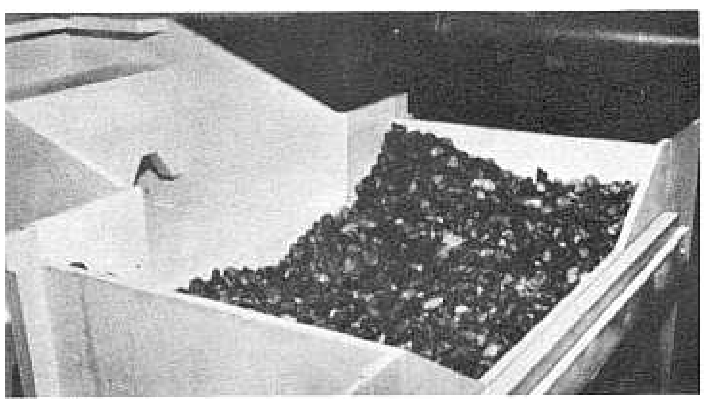
Figure 6.–Entrance pipe flowing half full with uncontrolled tailwater in 2.4-foot (730-mm) wide basin.



F = 3.53
W/D = 5.98
Minor erosion
Satisfactory



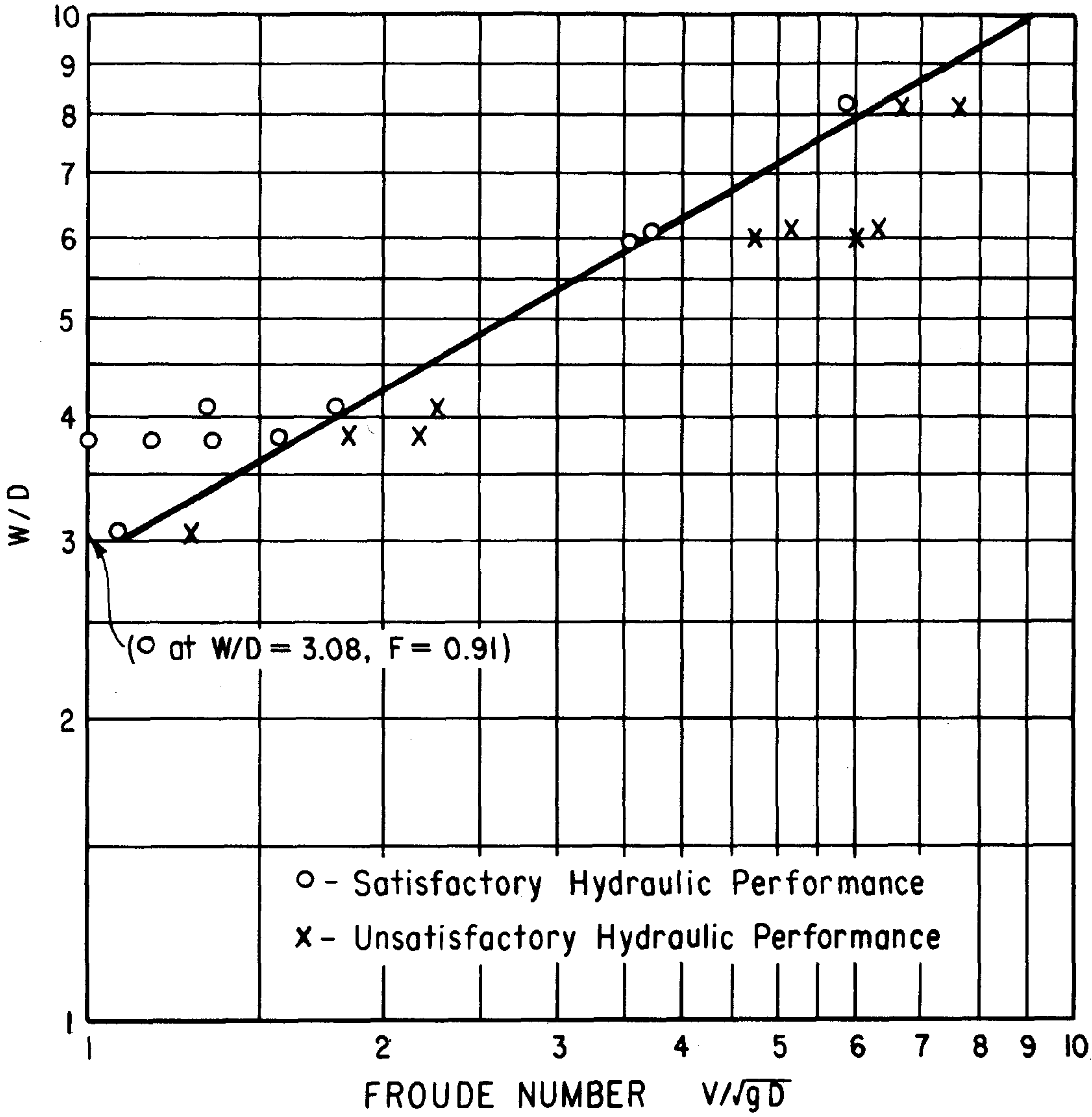
F = 4.77
W/D = 5.98
Excessive erosion
Unsatisfactory



F = 6.01
W/D = 5.98
Excessive erosion
Unsatisfactory

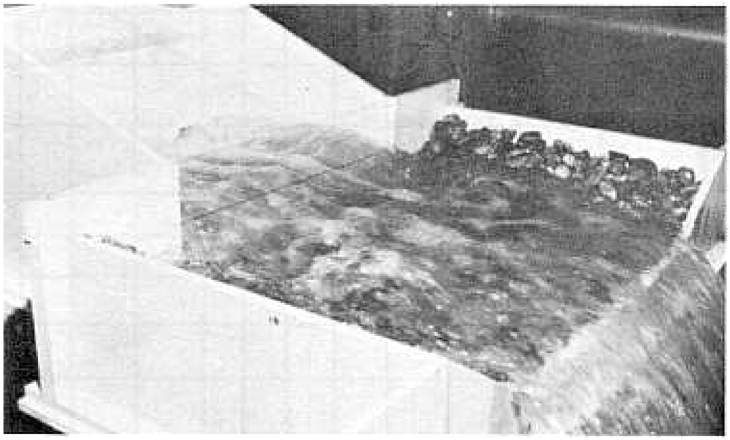
Note: For plot of these operating conditions, see figure 8.

Figure 7.–Erosion for uncontrolled tailwater with entrance pipe flowing half full in 2.4-foot (730-mm) wide basin.

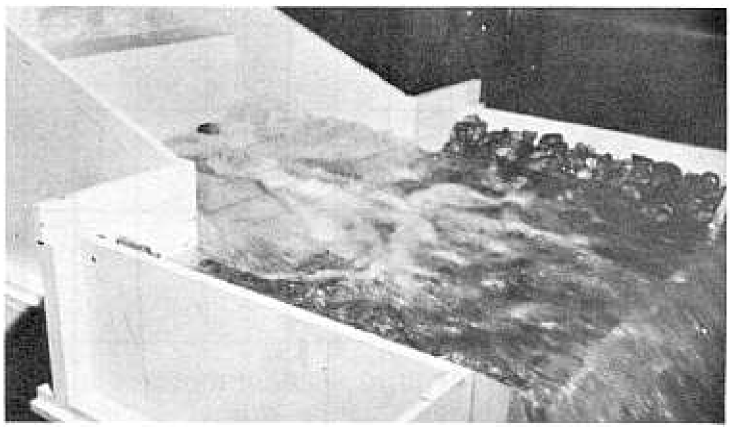


"W" is the inside width of the basin.
"D" represents the depth of flow entering the basin and is the square root of the flow area.
"V" is the velocity of the incoming flow.
The tailwater depth is uncontrolled.

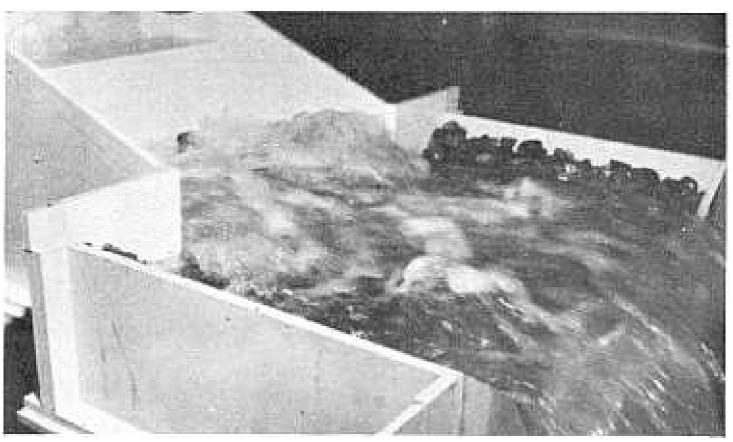
Figure 8.–Design width of basin.



F = 5.87
W/D = 8.15
No erosion
Satisfactory



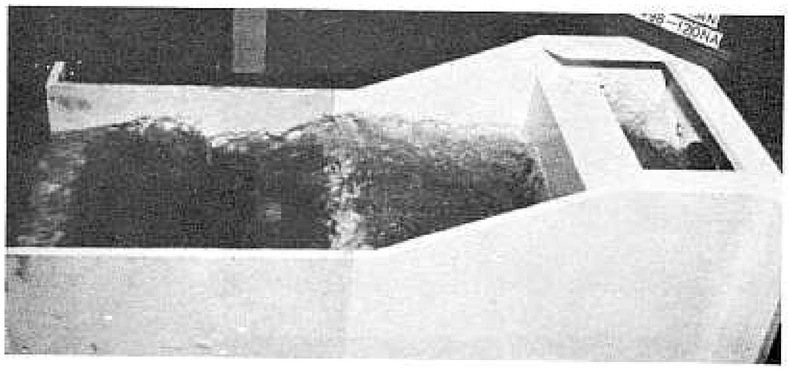
F = 6.67
W/D = 8.15
No erosion
Satisfactory



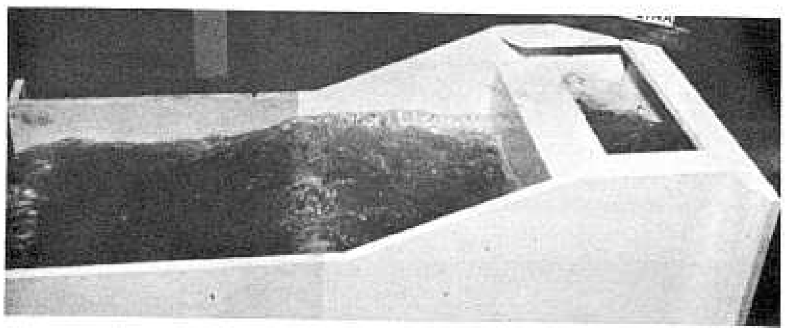
F = 7.59
W/D = 8.15
Minor erosion
Unsatisfactory

Note: For plot of these operating conditions, see figure 8.

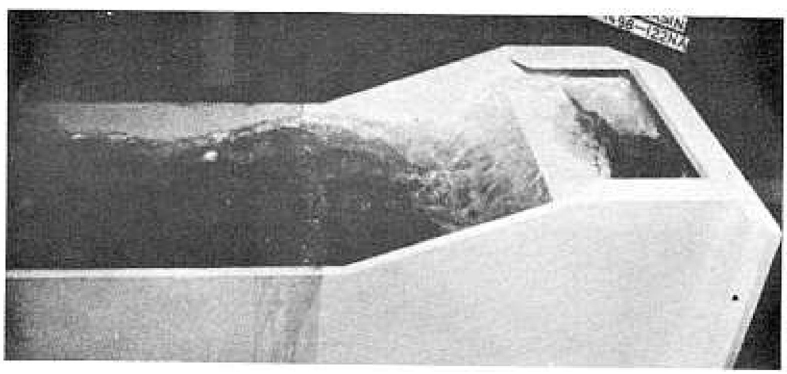
Figure 9.–Entrance pipe flowing one-fourth full with uncontrolled tailwater in 2.4-foot (730-mm) wide basin.



F = 0.91
W/D = 3.08
No erosion
Satisfactory



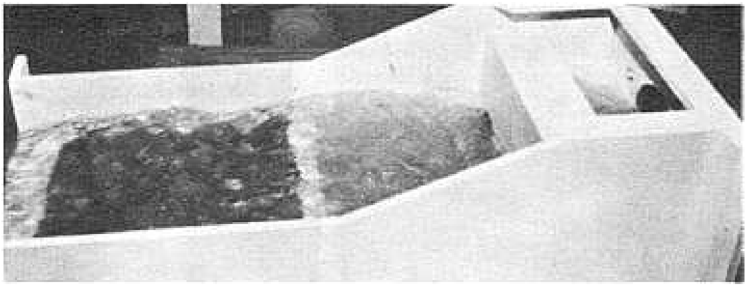
F = 1.07
W/D = 3.08
No erosion
Satisfactory



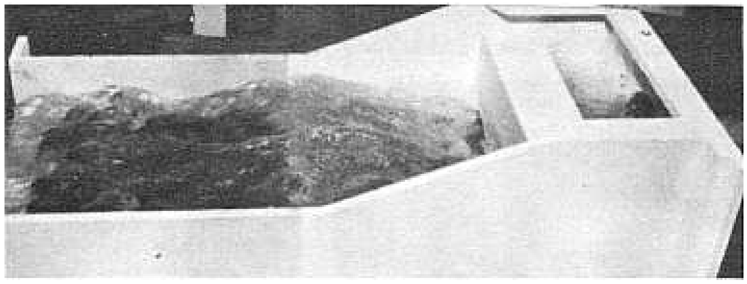
F = 1.27
W/D = 3.08
Minor erosion
Unsatisfactory

Note: For plot of these operating conditions, see figure 8.

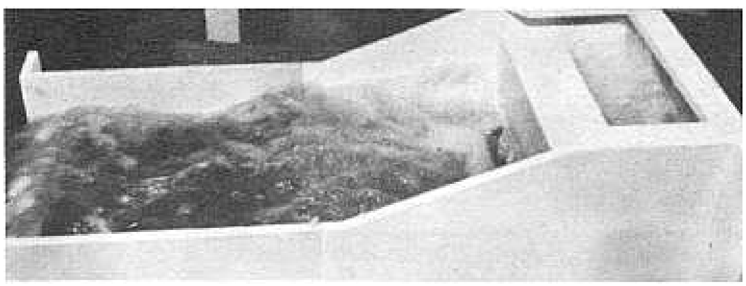
Figure 10.–Entrance pipe flowing three-fourths full with uncontrolled tailwater in 1.6-foot (490-mm) wide basin.



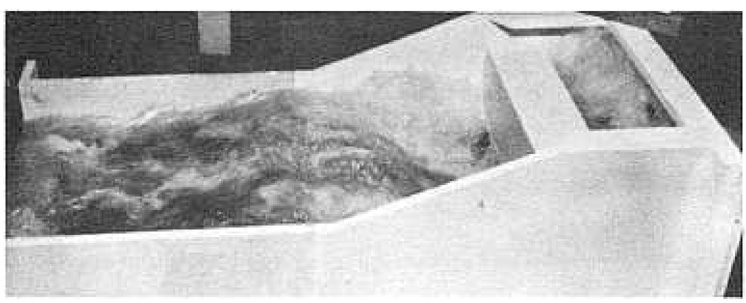
F = 1.01
W/D = 3.81
No erosion
Satisfactory



F = 1.56
W/D = 3.81
No erosion
Satisfactory



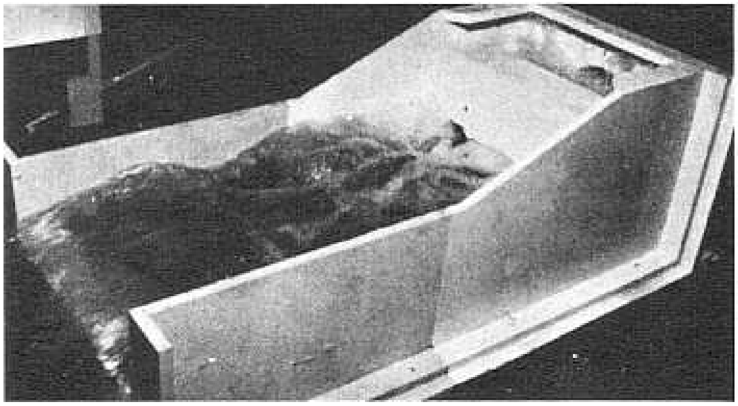
F = 1.83
W/D = 3.81
Minor erosion
Unsatisfactory



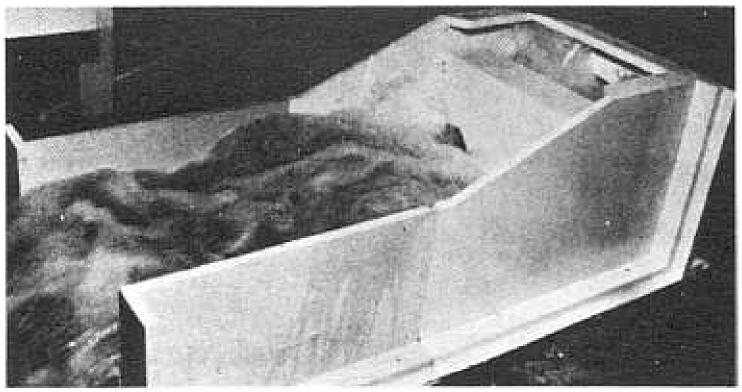
F = 2.17
W/D = 3.81
Excessive erosion
Unsatisfactory

Note: For plot of these operating conditions, see figure 8.

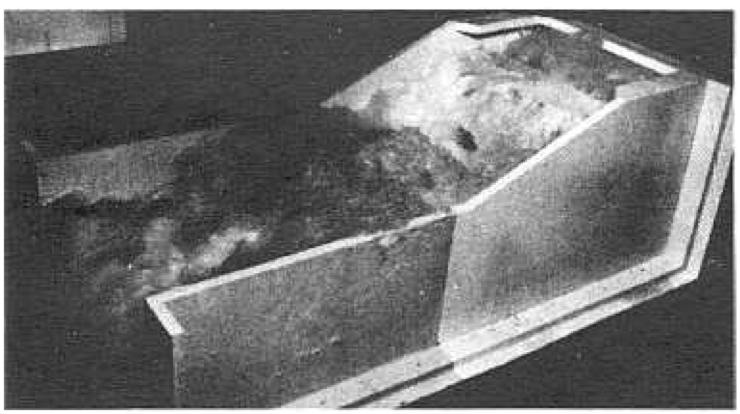
Figure 11.–Entrance pipe flowing half full with uncontrolled tailwater in 1.6-foot (490-mm) wide basin.



F = 3.72
W/D = 6.14
No erosion
Satisfactory



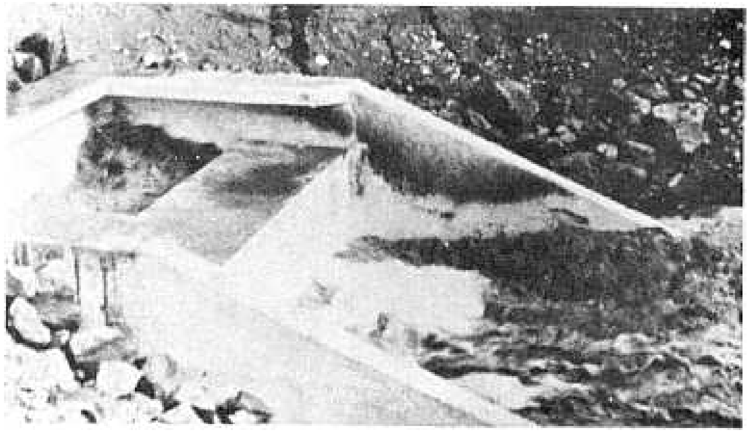
F = 5.11
W/D = 6.14
Excessive erosion
Unsatisfactory



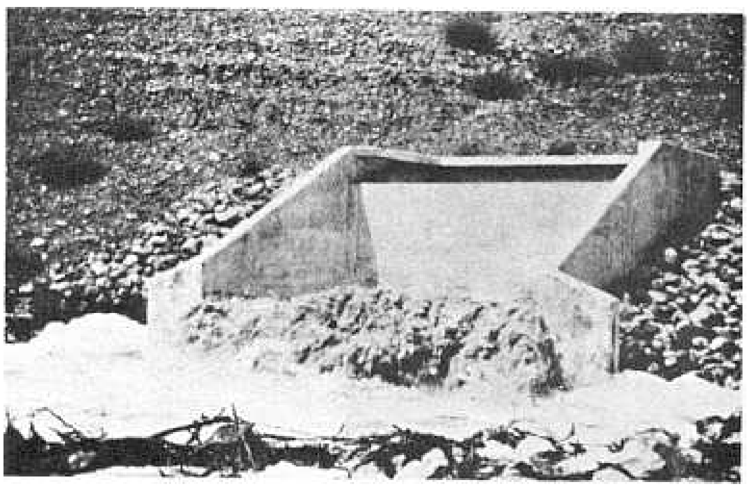
F = 6.28
W/D = 6.14
Excessive erosion
Unsatisfactory

Note: For plot of these operating conditions, see figure 8.

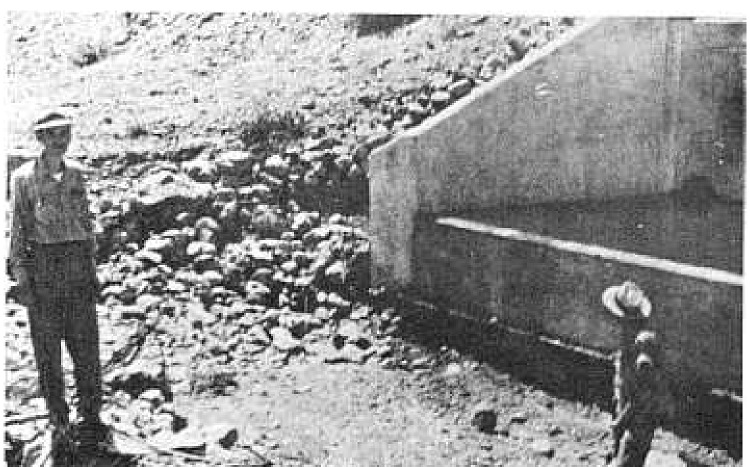
Figure 12.–Entrance pipe flowing one-fourth full with uncontrolled tailwater in 1.6-foot (490-mm) wide basin.



Picacho South Dam outlet works structure discharging 130 ft^3/s (3.7 m^3/s) (80 percent of maximum capacity).



Picacho North Dam outlet works structure discharging 210 ft^3/s (5.9 m^3/s) (80 percent of maximum capacity).



Scour below Picacho North Dam outlet works following flood of August 20, 1954. Evidence points to undersized riprap.

Note: At full capacity the basins are approximately 13 percent undersized, based on present design standards.

Figure 13.–Prototype operation.

Standardization of the Discharge Channel Riprap

Channel bed erosion tests were not conducted to prove the required size of stones in the riprap. Instead, a reasonable riprap size was chosen to fit the size of the basin. Having predetermined the basin size and relative size of stones in the riprap, the discharge capacity and entrance velocity limitations were determined as already described.

A model riprap was chosen that approximated a basin width-to-stone diameter ratio of 20 to 1. This size appeared to be reasonable and satisfactory, as was confirmed by the tests described in the preceding section on standardization of basin size. These tests showed that slight erosion of the riprap began at about the same time as excessive water surface roughness appeared within and downstream of the basin. The model stones were rounded, although angular ones would be preferred in the prototype.

The gravel was placed on the channel bottom at end still elevation and on the 1-1/2 to 1 side slopes to a normal depth equal to the height of the end sill (fig. 1) and for a distance downstream equal to the basin width. This arrangement was satisfactory in the model tests and is, therefore, recommended for prototype construction.

In some instances, the discharge channel bed may be several inches or a few feet below the end sill elevation. This will considerably increase the riprap stone size requirement. To determine the increased riprap stone size requirement, the average flow velocity at the end sill was determined. It was then related to the average entrance velocity and plotted versus Froude number in figure 14, and plotted versus the stone size requirement in figure 15.

The additional head as provided by the lower channel bed should be added to the velocity head at the sill to determine the velocity of flow entering the channel. Having determined the increased velocity, figure 15 can be entered to determine the riprap stone size requirement.

The stone size requirement for end sill velocities is compared in figure 15 with the stone size requirement for bottom velocities in channels downstream of stilling basin [*2*]. The comparison indicates that the stone size recommendation here is conservative; however, the flow from the sill is in a downward direction as there is a drop in water surface from end sill to channel (fig. 14). Also, the average velocity plotted in figure 14 is not as high as the velocity of flow from the center of the sill.

Tailwater Recommendations

The effect of tailwater on the basin efficiency was determined by repeating the above tests using a maximum tailwater controlled to a depth of $d+b/2$ above the basin floor. [*2*] (See fig. 1 for definitions.) A comparison of these flow conditions (figs. 16 and 17) with the uncontrolled tailwater flow conditions (figs. 4 and 6) shows that the water surface roughness and bed erosion are reduced by the higher tailwater but not sufficiently to allow a reduction in the basin size. The riprap stone size could be reduced slightly as determined by the reduced velocity using figure 15.

Performance Evaluation

Energy dissipation is initiated by flow striking the vertical hanging baffle and being turned upstream by the horizontal portion of the baffle and by the floor, in vertical eddies. Its effectiveness is best illustrated by plotting the percent of energy loss between the entrance portal and the end sill for a range of operating conditions as represented by the Froude number (fig. 14). Comparing the energy loss with the losses in a hydraulic jump shows the impact basin to be more efficient.

Prototype structures [*2*] that meet these design standards have operated successfully. The outlet basins at Picacho South and North Dams, discharing at 80-percent capacity, are examples (fig. 13). The design requirements for the 80-percent capacity and for the 100-percent design capacity are given in table 1.

For operation of these structures at 80-percent capacity, the table shows the width of basin and, therefore, the size of basins to be adequate to meet design requirements. However, for 100-percent design capacity, the table shows the basins to be about 13 percent undersized based on the design standards presented herein (fig. 8). The actual performance proved this to be true (fig. 13).

The prototype structures at Picacho South and North Dams can also be used to verify the recommended size of riprap. According to construction specifications for both dams, the riprap below the outlets was to "* * * consist of durable rock

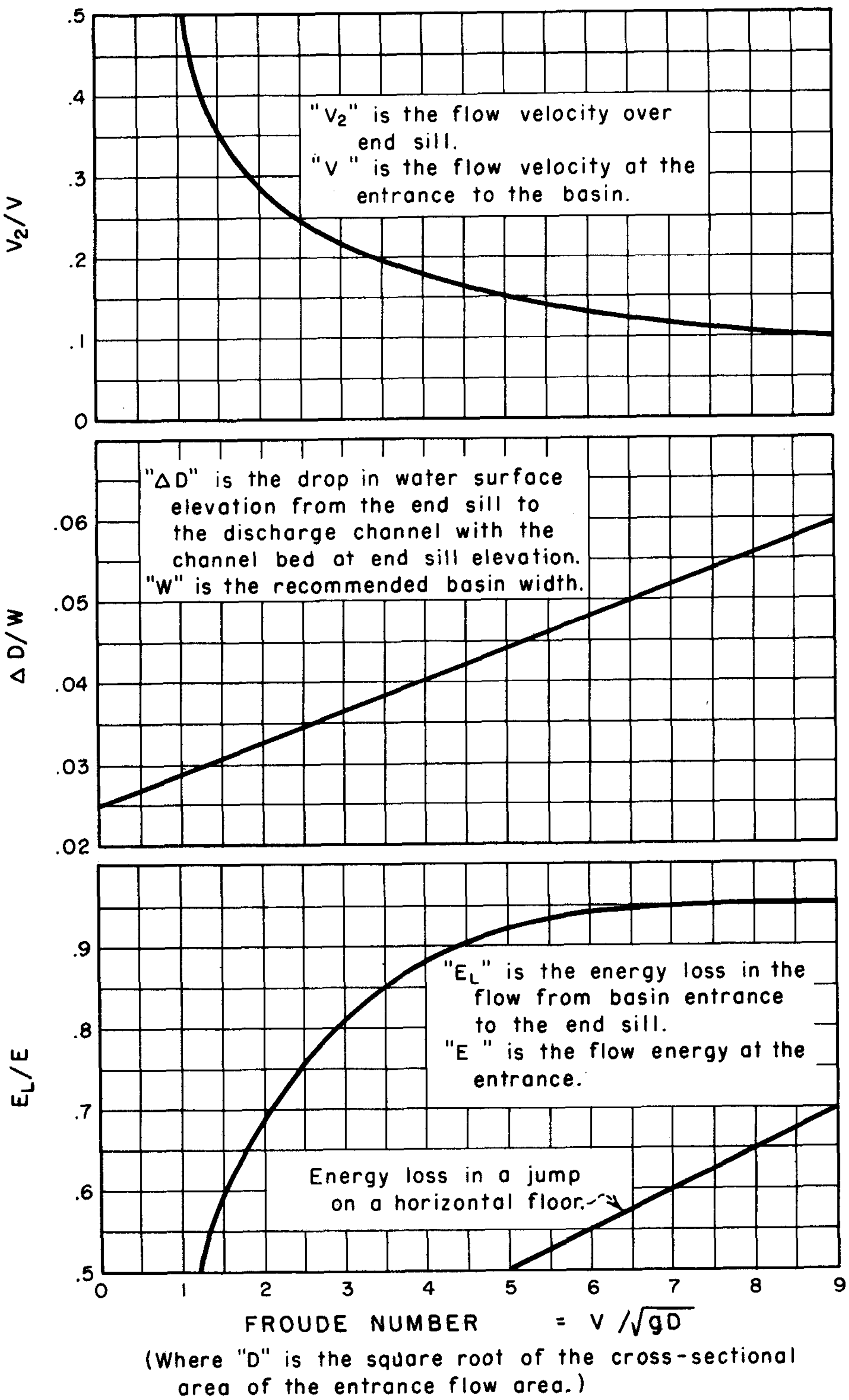
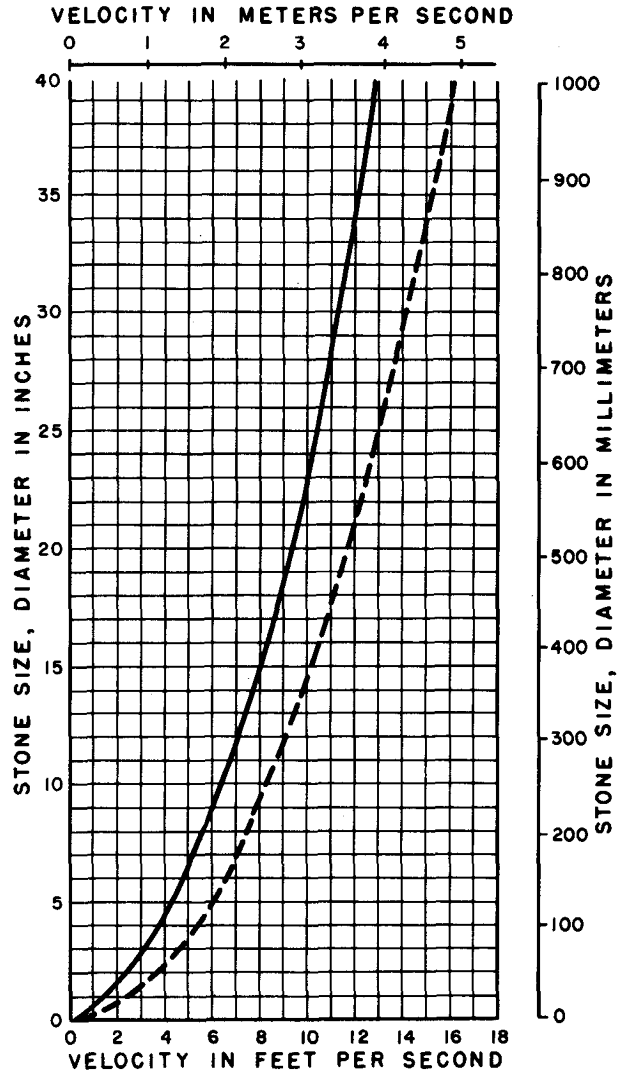


Figure 14.–End sill velocity, water surface drop from end sill, and energy loss through basin.

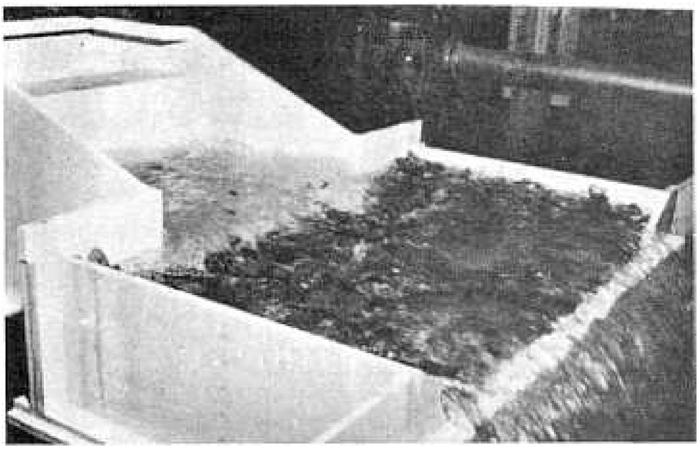


Note: The riprap should be composed of a well-graded mixture but most of the stones should be of the size indicated by the curve.

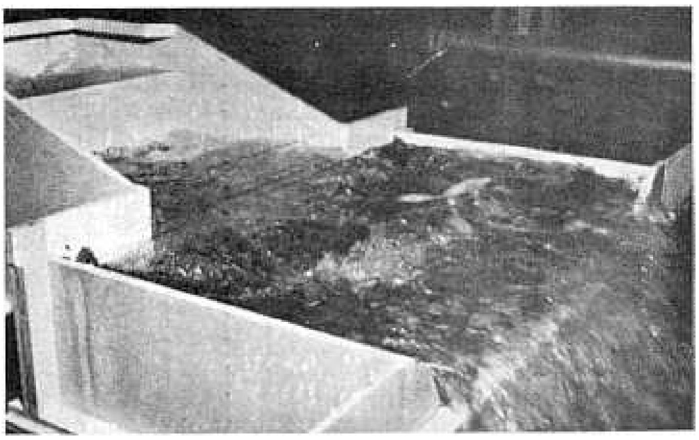
— End sill velocity in type VI Basin vs stone size required in riprap.

---- Bottom velocity in a channel vs stone size required in riprap. (See figure 165 in reference 2)

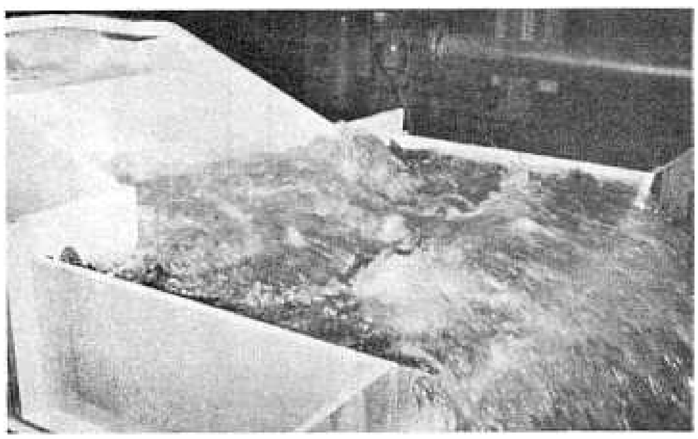
Figure 15.–Recommended riprap stone size.



F = 1.34
W/D = 4.06
No erosion
Satisfactory



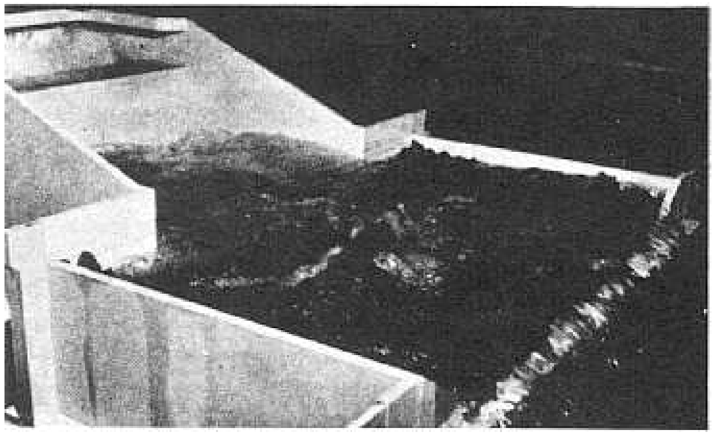
F = 1.81
W/D = 4.06
No erosion
Satisfactory



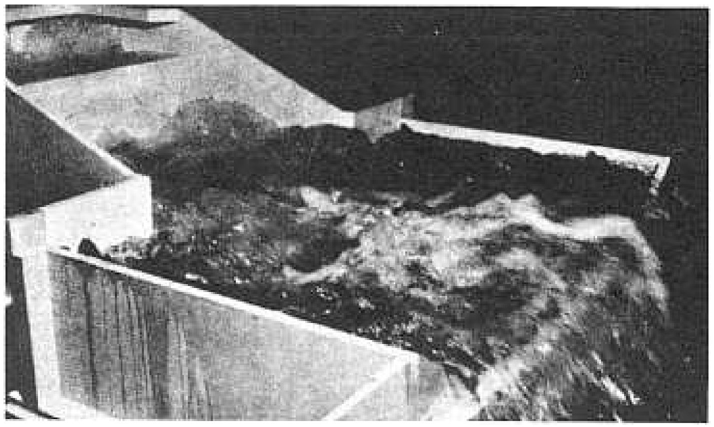
F = 2.27
W/D = 4.06
Excessive erosion
Unsatisfactory

Note: Tailwater = d + b/2; see figure 1 for definitions.

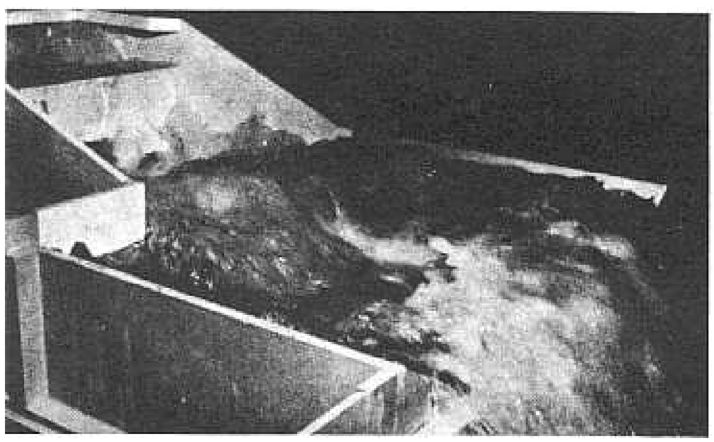
Figure 16.–Entrance pipe flowing full with controlled tailwater in 2.4-foot (730-mm) wide basin.



F = 3.53
W/D = 5.98
No erosion
Satisfactory



F = 4.77
W/D = 5.98
Minor erosion
Unsatisfactory



F = 6.01
W/D = 5.98
Excessive erosion
Unsatisfactory

Note: Tailwater = d + b/2; see figure 1 for definitions.

Figure 17.–Entrance pipe flowing half full with controlled tailwater in 2.4-foot (730-mm) wide basin.

fragments reasonably graded in size * * *" from 3.4 cubic feet (95 dm^3) to 0.1 cubic foot (3 dm^3). The individual rocks would range from about 18- to 5.5-inch (460- to 140-mm) cubes and have a mass of 500 to 15 pounds (225 to 7 kg). Although it is impossible from the photograph of the outlet at North Dam (fig. 13) to determine the size of stones in the channel riprap at the start of the run, the bank riprap indicates that there were very few pieces of the 500-pound (225-kg) size. The few remaining pieces near the man at the right seem to be in the upper range of sizes and are not washed out. It is also difficult to determine the elevation of the channel bed at the beginning of the run; but, here again, the bank riprap and the waterfall effect of the flow over the end sill in figure 13 indicate that there is a drop from the end sill to the channel, as shown in the table. Therefore, the majority of the stones in the riprap should be 28 inches (710 mm) in diameter as recommended here. Since the specified stones were smaller than this size, the riprap would be expected to fail and did.

At South Dam, the photographs of the outlet discharging do not show a waterfall effect from the end sill. Therefore, the riprap was probably nearer to end sill elevation than specified in the table. This would reduce the required stone diameter to something less than 18 inches (460 mm), but greater than 8 inches (200 mm). Since this range is within that specified, the riprap would be expected to remain in place and did.

Alternate End Sill Design

The alternate end sill design (fig. 1) having 45° wingwalls was not tested in this study. Examination of the data and photographic results of the earlier studies [*2*], however, indicated that height of boil and drop in water surface elevation to the channel (fig. 14) will be reduced by using the 45° wingwalls and a longer end sill. The use of this sill would allow the flow to spread more uniformly over a wider channel and, thereby, reduce erosion tendencies and wave heights.

Debris Barrier and Trashrack

At some prototype installations, weeds and debris such as Russian thistles have been trapped in the basin between the pipe portal and the baffle. This debris has compacted to the extent of blocking the portal, thus reducing the capacity of the structure. The compacted weeds will not wash out and are very difficult to remove. The only satisfactory field method of removing the debris has been to destroy portions of the baffle.

This condition was tested in the two models using Russian thistle branches. The model demonstrated that the thistles would not wash out and no satisfactory method of making the basin self-cleaning of weeds and debris was developed.

At structures where thistles or other debris are likely to be a problem, it is suggested that screening be used to cover the upstream portion of the basin and that a screen or trashrack device be used where the flow enters the pipe to the basin.

Self-Cleaning Feature

Sediment may accumulate in the basin below the hanging baffle during periods of nonuse. The notches were installed in the baffles to provide an opening through which a jet would discharge to begin erosion and removal of the sediment from the basin.

The 2.4-foot (730-mm) wide basin was operated with the portion of the basin below the hanging baffle blocked to simulate a sediment-filled basin. It was determined from this test that the design discharges could be passed over the top of the baffle with very little splashing outside the basin and, in general, only minor erosion in the riprapped area. This type of operation could be tolerated for a limited time while sediment is being washed from the basin. If it is anticipated that the basin beneath the baffle will remain relatively free of sediment, the notches may be omitted.

Table 1.—*Design specifications for the outlet works structure at Picacho North and South Dams*

	Estimated maximum flood of record		Maximum design flood	
	South Dam	North Dam	South Dam	North Dam
Discharge (Q)	130 ft^3/s (3.7 m^3/s)	210 ft^3/s (5.9 m^3/s)	165 ft^3/s (4.7 m^3/s)	275 ft^3/s (7.8 m^3/s)
Estimated entrance velocity (V)	31.8 ft/s (9.7 m/s)	37 ft/s (11.3 m/s)	39 ft/s (11.9 m/s)	48 ft/s (14.6 m/s)
Cross-sectional area of flow (A)	4.09 ft^2 (0.38 m^2)	5.67 ft^2 (0.527 m^2)	4.23 ft^2 (0.393 m^2)	5.73 ft^2 (0.532 m^2)
Depth (D)	2.02 ft (0.616 m)	2.38 ft (0.725 m)	2.06 ft (0.628 m)	2.39 ft (0.728 m)
Froude number (F)	3.94	4.23	4.77	5.46
Width to depth ratio (W/D), figure 8	6.21	6.50	6.95	7.50
Width recommended (W)	12.54 ft (3.82 m)	15.47 ft (4.72^- m)	14.31 ft (4.36 m)	17.93 ft (5.47 m)
Width actually used	12.50 ft (3.81 m)	15.50 ft (4.72^+m)	12.50 ft (3.81 m)	15.50 ft (4.72^+m)
Percent undersized	<1	NA	13	13
Ratio of end sill velocity to entrance velocity (V_2/V)	0.18	0.17	0.158	0.149
End sill velocity in recommended width basin (V_2)	5.72 ft/s (1.74 m/s)	6.29 ft/s (1.92 m/s)	6.16 ft/s (1.88 m/s)	6.77 ft/s (2.06 m/s)
Velocity head in end sill	0.50 ft (0.15 m)	0.61 ft (0.19 m)	0.58 ft (0.18 m)	0.71 ft (0.22 m)
Drop from end sill to channel (y)	0.75 ft (0.23 m)	1.33 ft (0.41 m)	0.75 ft (0.23 m)	1.33 ft (0.41 m)
Velocity head in channel	1.25 feet (0.38 m)	1.94 ft (0.59 m)	1.33 ft (0.41 m)	2.04 ft (0.62 m)
Velocity in channel	8.98 ft/s (2.74 m/s)	11.17 ft/s (3.40 m/s)	9.26 ft/s (2.82 m/s)	11.47 ft/s (3.50 m/s)
Riprap stone diameter (fig. 15) for recommended basin above channel	18 in (460 mm)	28 in (710 mm)	20 in (500 mm)	30 in (760 mm)
Riprap stone diameter (fig. 15) for channel at end sill elevation	8 in (200 mm)	9.8 in (250 mm)	9.3 in (240 mm)	11 in (280 mm)
Stone diameter specification, both dams	18 to 5.5 inches (460 to 140 mm)			

DESIGN CONCLUSIONS AND RECOMMENDATIONS

The following procedures and rules are recommended in the design of the type VI basin:

1. Given a design discharge "Q," determine the velocity "V" and Froude number "F" of the incoming flow. If the Froude number is more than 10, use of this basin is not practicable. In computing the Froude number, assume the depth "D" to be the square root of the cross sectional area of the flow at the entrance "Q/V."

2. The flow is usually from a pipe. If the pipe flows partially full, it should be vented at the upstream end.

3. If the entrance pipe slopes downward, the outlet end of the pipe should be turned horizontal, or the invert filled to form a horizontal surface, for at least one pipe diameter upstream from the portal. For slopes 15° or greater, the horizontal length of pipe or fillet should be two or more diameters.

4. If the flow enters the basin from a rectangular open channel, the channel walls should be as high as the basin walls and the invert should be horizontal for a minimum of two channel widths upstream from the basin.

5. Having determined the Froude number, enter figure 8 to find the minimum required width of basin.

6. Figure 8 shows data points above the recommended width that provides satisfactory operation for basins larger then the design limit; however, if the basin is too large, the incoming jet will pass under the hanging baffle to reduce the effectiveness of the basin. Since the basin will be larger than need be for less than design flows, the basin should not be oversized for the design flow.

7. Relate the basin dimensions to the basin width in accordance with figure 1. The dimension "t" is a suggested minimum thickness for the hanging baffle and is not related to the hydraulic performance of the structure.

8. To prevent the possibility of cavitation or impact damage to the basin, the entrance velocity should be limited to about 50 feet per second (15 m/s).

9. Riprap with a well-graded mixture of stones, most of which have diameters equal to 5 percent of the basin width, should be placed to a depth equal to the height of end sill for a distance equivalent to one basin width downstream from the end sill.

If the elevation of the channel bed is below the end sill, the velocity of flow entering the channel will be increased and the riprap stone size should be increased as determined using figure 15. The drop in elevation from sill to bed must be added to the velocity head of the flow at the end sill, as determined from figure 14, to obtain the average velocity of flow entering the tailwater channel. This velocity can be used in figure 15 to determine the size of stones required.

10. Tailwater depth other than that created by the natural slope of the channel is not required. However, a smoother water surface will be obtained and smaller riprap stones can be used by increasing the tailwater depth in the channel to a depth of $d+b/2$ (see fig. 1 for definition of "d" and "b") above the basin floor. Compare figures 4 and 6 with figures 16 and 17.

11. This basin is more effective in the dissipation of energy than the hydraulic jump, figure 14. Prototype basins have operated successfully with entrance velocities up to 38 feet per second (11.6 m/s) (table 1 and fig. 13), and the recommended riprap size requirement has been verified by the performance of these basins.

12. The alternate end sill design (fig. 1) utilizing the 45° wingwall is not required but will reduce the drop in water surface elevation from end sill to channel (fig. 14) and reduce channel erosion.

13. No practical method of making the basin self-cleaning of debris, such as Russian thistles, was found. Where debris is a problem, screening devices are recommended at the entrance to and over the top of the structure. If thistles are allowed to enter the basin, they will not wash out.

14. During periods of nonoperation, sediment may accumulate in the basin. Notches in the baffle (fig. 1) are recommended to provide two jets that will start the erosion of the sediment which will eventually be washed from the basin. However, the basin is capable of satisfactorily discharging the entire design flow over the top of the baffle for short periods of time.

REFERENCES

[*1*] "Progress Report II–Research Study on Stilling Basins, Energy Dissipators, and Associated Appurtenances," Bureau of Reclamation Report No. Hyd-399 (1955).

[*2*] "Hydraulic Design of Stilling Basins and Energy Dissipators," Bureau of Reclamation Engineering Monograph No. 25 (1963).

ABSTRACT

Model studies on 1.6- and 2.4-ft-wide (490 and 730 mm) type VI stilling basins were conducted to modify existing standard design procedures. Investigations were concerned with: basin entrance flow conditions including type of entrance, slope, velocity, and Froude number; basin dimensions in relation to the basin width; basin width in relation to Froude number; and riprap size and location. Performance was evaluated in terms of energy dissipation and prototype operation. An optimum tailwater, an alternate end sill design, methods of preventing clogging of the basin, and means for automatic removal of sediment from the basin were suggested.

DESCRIPTORS–/ *stilling basins/ entrances/ *riprap/ erosion/ *hydraulic models/ hydraulic structures/ discharges/ *energy dissipation/ velocity/ pipes/ open channels/ debris barriers/ *laboratory tests/ baffles/ model tests/ sediment concentration/ trashracks/ impact.

IDENTIFIERS–/ deflectors/ Franklin Canal, Tex/ *energy dissipators/ progress reports.

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