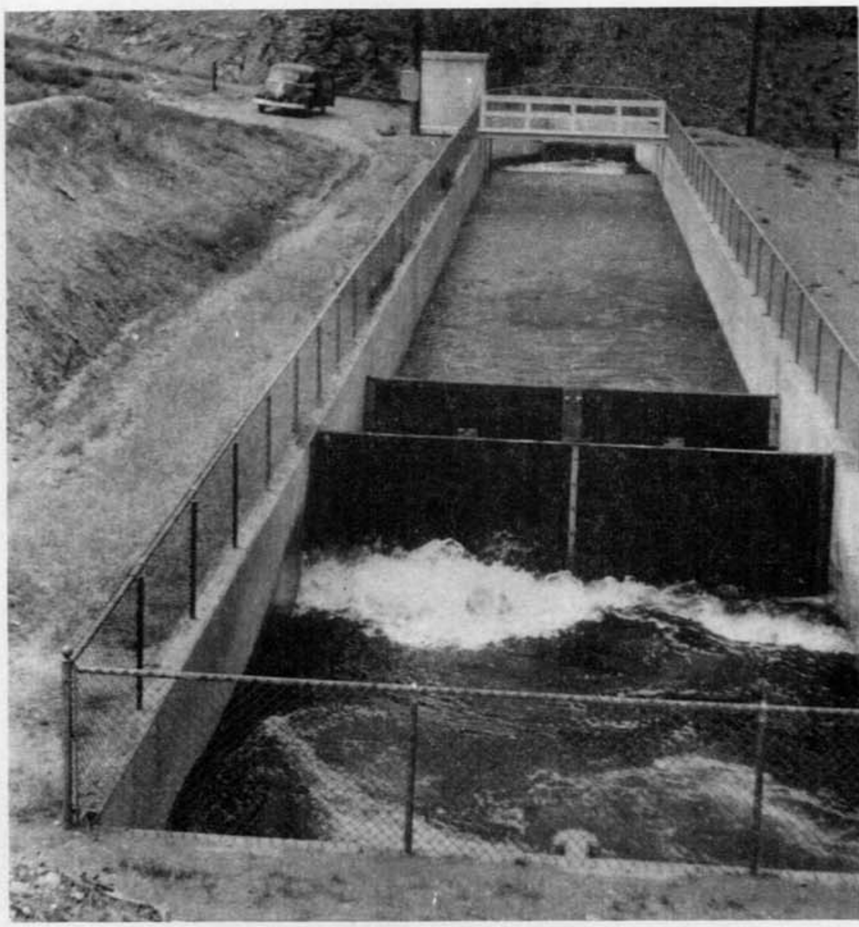
Department of the Interior

Bureau of Reclamation

OPERATION AND MAINTENANCE EQUIPMENT AND PROCEDURES

RELEASE NO. 14

October, November and December 1955



CONTENTS

Cover Photograph

Performance of wave suppressor--Flatiron Afterbay Dam Outlet Works--Colorado-Big Thompson Project, Colorado.

INTRODUCTION

This is Release No. 14 of the Operation and Maintenance Equipment and Procedures Bulletin and is being devoted to studies made on stilling basins, energy dissipators and associated appurtenances by the Bureau's Hydraulic Laboratory, located at the Denver Federal Center, Denver, Colorado. The full study made by Engineers J. N. Bradley and A. J. Peterka was published as Hydraulic Laboratory Report No. Hyd-399. The portion of the report condensed and given in this issue of the Bulletin includes only that part of the study devoted primarily to canal structures or necessary to understand the material on canal structures.

A forward to the report, by Engineer H. M. Martin, Chief of the Hydraulic Laboratory, explains the need for the study in the design of irrigation works and is quoted in part below:

> " * * * Since 'Stilling Basins' did not lend themselves to mathematical analysis, they became the center of discussion between laboratory and design personnel. It was realized that unexplainable gaps existed in the available information resulting in uncertainty, confusion, and sometimes apparent contradiction, when stilling basin designs were attempted without individual hydraulic model tests. To resolve these differences, to close the gaps, and to generalize the design of stilling basins, the laboratory's general investigation program for the past 2 years has included a coordinated program of stilling basin research.
>
> "As the study progressed and the outcome became increasingly promising, numerous requests for design criteria even in draft form were received. To satisfy this immediate demand, a tentative and limited edition of Hydraulic Laboratory Report No. Hyd-380 was issued. Comments and criticism were invited.
>
> "The immediate requirement having been temporarily satisfied, the tentative edition was next given a critical review and certain parts were rewritten for the sake of clarity. More information, along with more definite design limits, was rewritten into other parts. New material, gathered since the first publishing, has been added to meet some of the deficiences of the tentative edition. The written material has been broken down into more titled units to make the report more useful as a reference volume. To also aid in this respect, a pictorial summary of the six sections of the report has also been added as a Frontispiece. Section 4 has been entirely rewritten to include the most recent developments in wave suppressors for open channels. Section 6 is entirely new and presents an economical stilling

basin for use on small structures where tail water is nonexistent or indefinite."

The entire study as planned and given below has not been completed. Laboratory Report No. Hyd-399, designated "Progress Report II, Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances," is divided into six sections and covers the first 6 items of the planned program. The other items will be completed as time and funds permit. Items 3, 4, and 6 pertain to canal structures in general and are the items included in this issue of the Bulletin. For the benefit of those desiring additional detail and the technical analyses of the problems involved, report Hyd-399 can be obtained by writing the Assistant Commissioner and Chief Engineer, Bureau of Reclamation, Denver Federal Center, Denver, Colorado.

SCOPE

1. General investigation of the hydraulic jump on a horizontal apron.

2. Stilling basin with horizontal apron, utilizing chute blocks at the upstream end and a dentated sill at downstream end such as are often used on high dam and earth spillways. The appurtenances modify the jump, causing it to form in a shorter than normal length.

3. Unusually short type of stilling basin suitable for canal structures, small outlet works, and small spillways where baffle blocks are used to effect a further shortening of the jump.

4. Stilling basin, alternate basin, and two types of wave suppressors, for use on canal structures, outlet works, and diversion dams.

5. Stilling basin with sloping apron for large capacities and high velocities, where appurtenances in basin are undesirable.

6. Extremely short impact-type stilling basin for use on outlets where tail water is nonexistent or unknown.

7. Overchute type of dissipator where baffle blocks distributed over the entire length and width of the chute dissipate the energy in the water as it falls.

8. Stilling basin for diversion dams where temporary retrogression is expected.

9. Stilling basin for diversion dams which can accommodate both free and submerged flow.

10. Stilling basin for use on high head outlet works, utilizing hollow-jet valves.

11. Slotted bucket for medium and low overfall dams.

12. Solid bucket for overfall dams where an excess of tail water exists.

13. Flip bucket which discharges above the tail water.

The Operation and Maintenance Equipment Procedures Bulletin is published in the Commissioner's Office, Denver, Colorado. It is circulated for the benefit of irrigation project operation and maintenance people. Its principal purpose is to serve as a medium of exchanging operating and maintenance information. Reference to a trade name does not constitute an endorsement of a particular product, and omission of any commercially available item does not imply discrimination against any manufacturer. It is hoped that labor-saving devices or less costly equipment developed by the resourceful water users will be a step toward commercial development of equipment for use on irrigation projects in continued effort to reduce costs and increase operating efficiency.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

STILLING BASINS, ENERGY DISSIPATORS AND ASSOCIATED APPURTENANCES
(From Hydraulic Laboratory Report No. Hyd-399)

OPERATION AND MAINTENANCE
EQUIPMENT AND PROCEDURES
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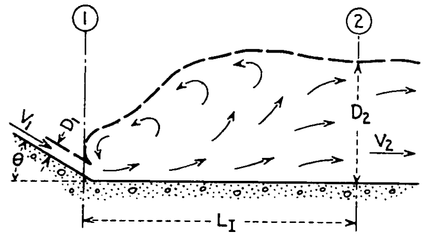
Denver, Colorado.
1955

STILLING BASIN I

NOTES

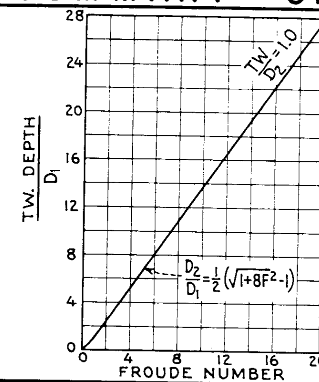
Jump occurs on flat floor with no chute blocks, baffle piers or end sill in basin. Usually not a practical basin because of expensive length.

Elements and characteristics of jumps for complete range of Froude numbers is determined to aid designers in selecting more practical basins II, III, IV, V and VI.



HYDRAULIC JUMP ON HORIZONTAL FLOOR

(FIGURE 4)



RATIO OF TW DEPTH TO D_1

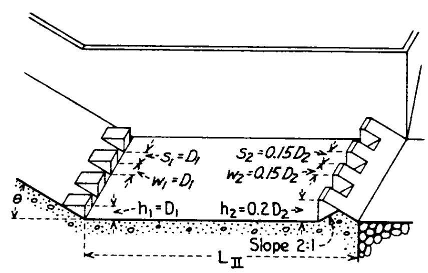
(FIG. 5)

STILLING BASIN II

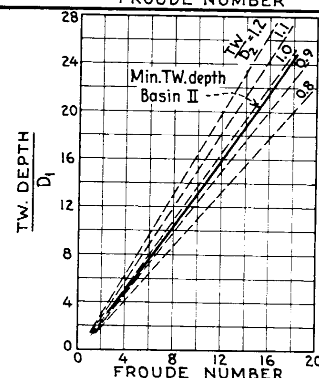
NOTES

Jump and basin length reduced about 33 percent with chute blocks and dentated end sill.

For use on high spillways, large canal structures, etc. for Froude numbers above 4.5.



(FIGURE 14)



MINIMUM TAILWATER DEPTHS (SWEEPOUT)

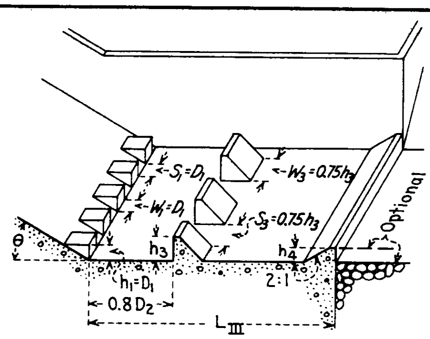
(FIG. 11)

STILLING BASIN III

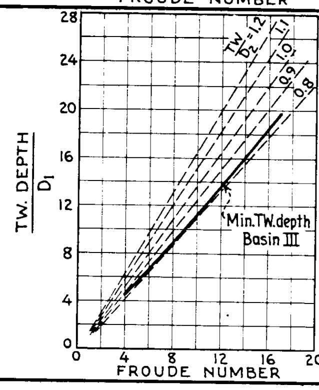
NOTES

Jump and basin length reduced about 60 percent with chute blocks, baffle piers, and solid end sill.

For use on small spillways, outlet works, small canal structures where V_1 does not exceed 50-60 feet per second and Froude number is above 4.5.



(FIGURE 17)



MINIMUM TAILWATER DEPTHS (SWEEPOUT)

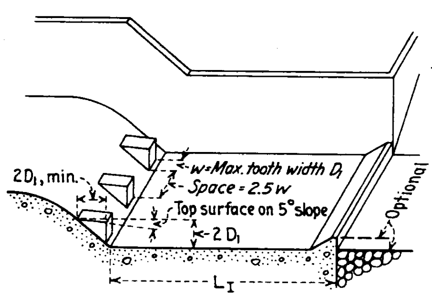
(FIG. 11)

STILLING BASIN IV

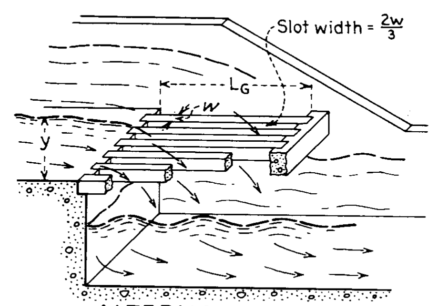
NOTES

For use with jumps of Froude number 2.5 to 4.5 which usually occur on canal structures and diversion dams. This basin reduces excessive waves created in imperfect jumps.

May also use alternate design and/or wave suppressors shown to right, or Basin VI



(FIGURE 22)



ALTERNATE DESIGN

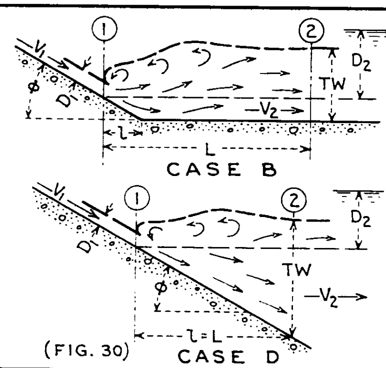
(FIGURE 23)

STILLING BASIN V

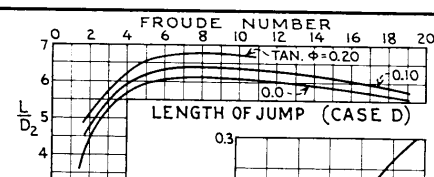
NOTES

For use where structural economies dictate desirability of sloping apron, usually on high dam spillways.

Needs greater tailwater depth than horizontal apron.



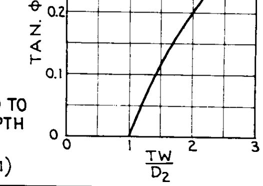
(FIG. 30) CASE D



(FIGURE 32)

TAILWATER DEPTH RELATED TO CONJUGATE DEPTH (CASE D)

(FIGURE 31)



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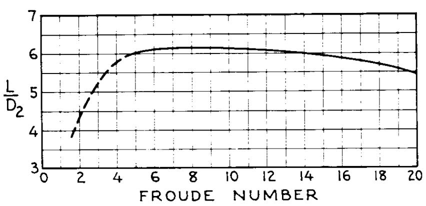
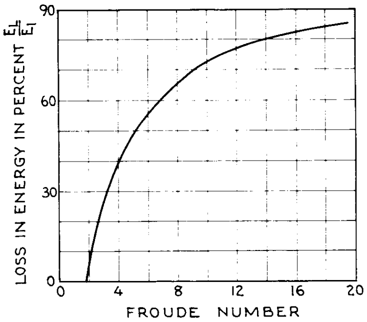
STILLING BASIN CHARACTERISTICS

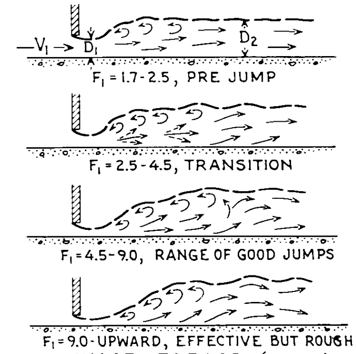


LENGTH OF JUMP

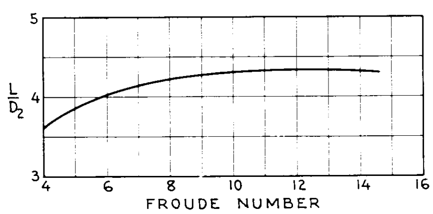
(FIGURE 7 OR 12)



LOSS OF ENERGY IN JUMP
(FIGURE 8)

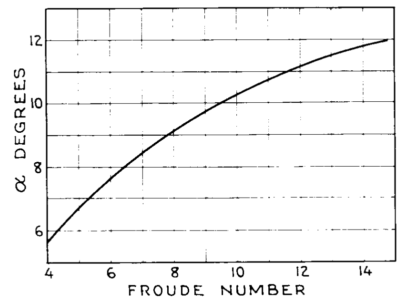


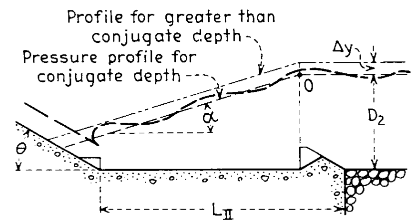
JUMP FORMS (FIG. 9)



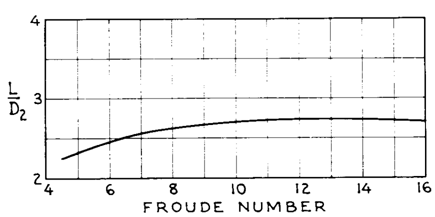
LENGTH OF JUMP

(FIGURE 12)



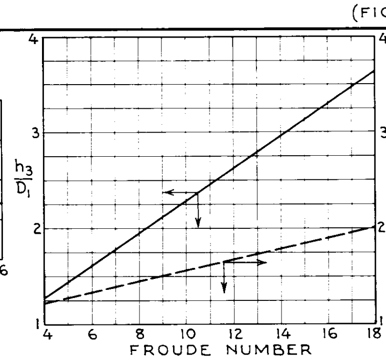


WATER SURFACE & PRESSURE PROFILES
(FIGURE 13)

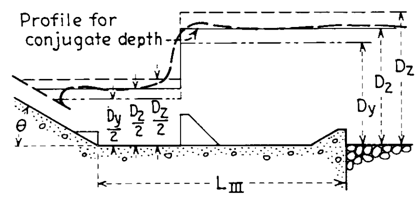


LENGTH OF JUMP

(FIGURE 12)

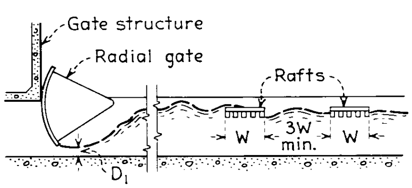


HEIGHT OF BAFFLE BLOCKS & END SILLS
(FIGURE 18)

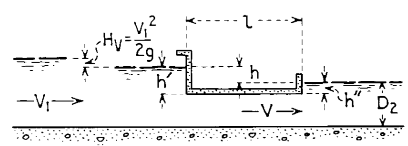


WATER SURFACE AND PRESSURE PROFILES

(FIGURE 19)



RAFT TYPE



UNDERPASS TYPE

WAVE SUPPRESSORS

(FIGURE 24) (FIGURE 29)

SUMMARY SHEET

NOTES

This sheet summarizes the main features discussed in this report, and shows some of the important charts. More charts are given in the report. This sheet should be used as a reference guide only; the entire report should be read before attempting to use any of the material contained herein.

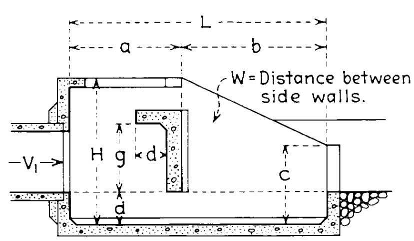
STILLING BASIN VI

NOTES

For use on pipe or open channel outlets, sizes and discharges from table. V_1 should not exceed 30 feet per second. No tailwater required. Froude number usually 1.5 to 7 but not important.

May substitute for Basin IV.

Energy loss greater than in comparable jump, figure. 44.



(FIGURE 42)

PIPE		Q	FEET AND INCHES							
DIA. IN.	AREA SQ.FT.		W	H	L	a	b	c	d	g
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	2-1
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	2-6
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	3-0
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	3-6
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	3-11
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	4-5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	4-11
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	5-4
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	6-2

(TABLE 11)

STILLING BASINS, ENERGY DISSIPATORS AND ASSOCIATED APPURTENANCES*

GENERAL

Stilling basins are defined as structures in which all or part of a hydraulic jump or other energy reducing action is confined. Other structures, such as buckets or impact dissipators, are designated energy dissipators.

Although the Bureau of Reclamation has designed and constructed hundreds of stilling basins and energy dissipation devices in conjunction with spillways, outlet works, and canal structures, it is still necessary in many cases to make model studies of individual structures to be certain that these will operate as anticipated. The reason for these repetitive tests, in many cases, is that a factor of uncertainty exists, which in retrospect is related to an incomplete understanding of the over-all characteristics of the hydraulic jump and other types of energy dissipators.

Previous laboratory studies made on individual structures over a period of years, by different personnel, for different groups of designers, with each structure having a different allowable design limitation for downstream erosion, resulted in a collection of data which on any plotting proved to be sketchy, inconsistent, and with only vague connecting links. Extensive library research into the works of others revealed the fact that these links were actually nonexistent.

To fill the need for up-to-date hydraulic design information on stilling basins and energy dissipators, the laboratory initiated a research program on this general subject. The program was begun with a rather academic study of the hydraulic jump, observing all phases as it occurs in open channel flow. With a broader understanding of this phenomenon, it was then possible to proceed to the more practical aspects of stilling basin design.

Existing knowledge, including laboratory and field tests collected from Hydraulic Laboratory records and experience over a 23-year period, was used to establish a direct approach to the practical problems encountered in hydraulic design. Hundreds of tests were also performed on both available and specially constructed equipment to obtain a fuller understanding of the data at hand and to close the

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*A condensation of Laboratory Report No. Hyd-399: Progress Report II; Research Study on Stilling Basins, Energy Dissipators, and Associated Appurtenances. Only the applicable Figures necessary for illustration are included in this condensation. They will be found in the Appendix and numbered as they appear in the original report to avoid confusion.

many loopholes. Testing and analysis were synchronized to establish valid curves in critical regimes, providing sufficient understanding of the hydraulic jump in its many forms to establish workable design criteria. Since all the test points were obtained by the same personnel, using standards established before testing began, and since results and conclusions were evaluated from the same datum, the data presented are consistent and reliable.

This report, therefore, is the result of the first integrated attempt to generalize the design of stilling basins, energy dissipators, and associated appurtenances. General design rules are presented so that the necessary dimensions for a particular structure may be easily and quickly determined, and the selected values checked by other designers without the need for exceptional judgment or extensive previous experience.

The report emphasizes design procedures rather than the hydraulic aspects of the data. Certain design procedures recommended in the past have been satisfactorily proved, while others have been modified or discarded in favor of improved methods. Satisfactory explanations are given for procedures, which in the past were considered inconsistent; for example, it is now fully understood why certain hydraulic jumps require a stilling basin only 2.5 times the downstream depth of flow while other jumps require basin lengths 4.5 times the depth of flow.

In most instances design rules and procedures are clearly stated in simple terms with limits fixed in a definite range. In other cases, however, it is necessary to state procedures and limitations in broader terms, making it necessary to carefully read the accompanying text.

Proper use of the material in report Hyd-399 will eliminate the need for hydraulic model tests on many individual structures, particularly the smaller ones. Structures obtained by following the recommendations will be conservative in that they will contain a desirable factor of safety. However, to further reduce structure sizes, to account for unsymmetrical conditions of approach or getaway, or to evaluate other unusual conditions not covered in this discussion, model studies will still prove beneficial.

Experimental Equipment

Five test flumes were used at one time or another to obtain the experimental data required in the present test program: Flumes A and B, Figure 1; Flumes C and D, Figure 2; and Flume F, Figure 3. The arrangement shown as Flume E, Figure 3, actually occupied a portion of Flume D during one stage of the testing, but it will be designated as a separate flume for ease of reference. Each flume served a useful purpose either in verifying similarity or extending the range of the experiments. Flumes A, B, C, D, and E contained overflow sections so that the jet entered the stilling basin at an angle to the horizontal. The degree of the angle varied in each case. In Flume F, the entering jet emerged from under a vertical slide gate so the initial velocity was horizontal.

The largest scale experiments were made on a glass-sided laboratory flume 4 feet wide and 80 feet long, in which an overfall crest with a slope of 0.8:1 was installed, Flume D, Figure 2. The drop from headwater to tail water in this case was approximately 12 feet, and the maximum discharge was 28 cfs.

It is felt that the design information to be presented will be found economical as well as effective, yet an effort was made to lean toward the conservative side. In other words, a moderate factor of safety has been included throughout. Thus, the information is considered suitable for general use with the following provision:

> It should be made clear at the outset that the information herein is based upon symmetrical and uniform action in the stilling basin and buckets. Should entrance conditions or appurtenances near the head of any of these structures tend to produce asymmetry of flow down the chute and in the stilling basin, these generalized designs may not be adequate. In this case, it may be advisable to make the basin in question of a more symmetrical nature, more conservative, or it may be wise to invest in a model study. Also, should greater economy be desired than these generalized designs indicate, a model study is recommended.

<u>Hydraulic Jump on Horizontal Apron</u> (Basin I)

A tremendous amount of experimental, as well as theoretical, work has been performed in connection with the hydraulic jump on a horizontal apron. There is probably no phase of hydraulics that has received more attention; yet, from a practical viewpoint there is still much to be learned.

As mentioned previously, the first phase of the present study was academic in nature, consisting of correlating the results of others and observing the hydraulic jump throughout its various phases; the primary purpose being to become better acquainted with the over-all jump phenomenon. The objectives in mind were: (1) to determine the applicability of the hydraulic jump formula for the entire range of conditions experienced in design; (2) as only a limited amount of information exists on the length of jump, it was desired to correlate existing data and extend the range of these determinations; and (3) it was desired to observe the various forms of the jump and to catalog and evaluate them.

Definitions of the symbols used in connection with the hydraulic jump on a horizontal floor are shown on Figure 4. The procedure followed in each test of this series was to first establish a flow. The tail water depth was then gradually increased until the front of the jump moved upstream to Section 1, indicated on Figure 4. The tail-water depth was then measured, the length of the jump recorded, and the depth of flow entering the jump, D_1, was obtained.

All computations are based on discharge per foot width of flume, or q.

The velocity entering the jump V_1 was computed by dividing q by D_1.

Froude Number

The Froude number is used throughout this presentation. The Froude number is simply:

$$F_1 = \frac{V_1}{\sqrt{gD_1}} \qquad (1)$$

where F_1 is a dimensionless parameter, V_1 and D_1 are velocity and depth of flow, respectively, entering the jump, and g is the acceleration of gravity. The law of similitude states that where gravitational forces predominate, as they do in open-channel phenomenon, the Froude number should be the same value in model and prototype. Although energy conversions in a hydraulic jump bear some relation to the Reynolds number, gravity forces predominate, and the Froude number is very useful for plotting stilling basin characteristics. As the acceleration of gravity is a constant, the term g could be omitted. Its inclusion makes the expression dimensionless, however, and the form shown as (1) is preferred.

The theory of the hydraulic jump in horizontal channels has been treated thoroughly by others and will not be repeated here. The expression for the hydraulic jump, based on pressure-momentum, occurs in many forms. The following form is most commonly used in the Bureau.

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2V_1^2 D_1}{g}} \qquad (2)$$

This may also be written:

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2V_1^2 D_1^2}{gD_1}}$$

Carrying D_1 over to the left side of the equation and substituting F_1^2 for $\frac{V_1^2}{gD_1}$,

$$\frac{D_2}{D_1} = -1/2 + \sqrt{1/4 + 2F_1^2}$$

or

$$\frac{D_2}{D_1} = 1/2 \left(\sqrt{1 + 8F_1^2} - 1 \right) \qquad (3)$$

Expression (3) shows that the ratio of conjugate depths is strictly a function of the Froude number. The ratio $\frac{D_2}{D_1}$ is plotted with respect to the Froude number on Figure 5. The line, which is virtually straight except for the lower end, represents the above expression for the hydraulic jump; while the points are experimental. The agreement is quite good for the entire range. There is an unsuspected characteristic, however, which should be mentioned.

Although the tail-water depth was sufficient to bring the front of the jump to Section 1 (Figure 4) in each test, the ability of the jump to remain at Section 1 for a slight lowering of tail-water depth became more difficult for the higher and lower values of the Froude number. The jump was least sensitive to variation in tail-water depth in the middle range, or values of F_1 from 4.5 to 9.

In cases where chutes or overfalls were used, the front of the jump was held near the intersection of the chute and the horizontal floor, as shown on Figure 4. The length of jump was measured from this point to a point downstream where either the high-velocity jet began to leave the floor or to a point on the surface immediately downstream from the roller, whichever was the longer. In the case of Flume F, where the flow discharged from a gate onto a horizontal floor, the front of the jump was maintained just downstream from the completed contraction of the entering jet. The point at which the high-velocity jet begins to rise from the floor is not fixed, but tends to shift upstream and downstream. This is also true of the roller on the surface. The ratio of length of jump to the conjugate tail-water depth D_2 is plotted with respect to the Froude number on Figure 7. This method of plotting was used throughout the study. The points represent the experimental values.

In addition to the curve established by the test points, curves representing the results of three other experimenters are shown on Figure 7. The greater portion of curve 1 is at variance with the present experimental results. Because of the wide use this curve has experienced, a rather complete explanation is presented regarding the disagreement. The curve 1 experiments were performed in a flume 6 inches wide, with limited head. The maximum discharge was approximately 0.7 cfs, and the thickness of the jet entering the jump, D_1, was 0.25 foot for a Froude number of 1.94. The results up to a Froude number of 2.5 are in agreement with the present experiments. The extreme case involved a discharge of 0.14 cfs and a value of D_1 of 0.032 foot, for $F_1 = 8.9$, which is much smaller than any discharge or value of D_1 used in the present experiments. Thus, it is reasoned that in the 6-inch-wide flume, frictional resistance in the channel downstream increased out of proportion to that which would have occurred in a larger flume or a prototype structure. In laboratory language, this is known as "scale effect" and is construed to mean that prototype action is not faithfully reproduced. The experimenters were somewhat dubious concerning the small scale experiments, and to confirm the above conclusion, it was found that results from Flume F, which was 1 foot wide, became erratic when the value of D_1 approached 0.10. Figure 7 shows three points obtained with a value of D_1 of approximately 0.085. The three points are given the symbol $\boxed{\times}$ and fall short of the recommended curve.

The two remaining curves, labeled 3 and 4, on Figure 7, portray the same trend as the curve obtained from the current experiments. The criterion used by each experimenter for judging the length of the jump is undoubtedly responsible for the displacement.

As can be observed from Figure 7, the test results from Flumes B, C, D, E, and F plot sufficiently well to establish a single curve. The five points from Flume A, denoted by squares, appear somewhat erratic and plot to the right of the general curve. Henceforth, reference to Figure 7 will concern only the recommended curve which is considered applicable for general use.

With the experimental information available, it is only a matter of computation to determine the energy absorbed in the jump. The total energy, E_1, entering the jump at Section 1 for each test is simply the depth of flow, D_1, plus the velocity head computed at the point of measurement. The energy leaving the jump is the depth of flow plus the velocity head at Section 2. E_L, the difference in the values, constitutes the loss of energy, in feet of water, attributed to the conversion. The ratio of energy lost in the jump E_L, to total energy entering jump, E_1, is plotted in percent against the Froude number and is shown as the curve to the left on Figure 8. For a Froude number of 2.0, which would correspond to a relatively thick jet entering the jump at low velocity, the curve shows the energy absorbed in the jump to be about 7 percent of the total energy entering. Considering the other extreme, for a Froude number of 19, which would be produced by a relatively thin jet entering the jump at very high velocity, the absorption by the jump would amount to 85 percent of the energy entering. Thus, the hydraulic jump can perform over a wide range of conditions. There are poor jumps and good jumps, with the most satisfactory occurring over the center portion of the curve.

Another method of expressing the energy absorption in a jump is to express the loss, E_L, in terms of D_1. The curve to the right on Figure 8, shows the ratio $\frac{E_L}{D_1}$ plotted against the Froude number. Losses in feet of head are obtained by this method. As there are those who prefer this method of plotting, the latter curve has been included.

The hydraulic jump may occur in at least four different distinct forms on a horizontal apron, as shown on Figure 9. Incidentally, all of these forms are encountered in design. The internal characteristics of the jump and the energy absorption in the jump vary with each form. Fortunately these forms, some of which are desirable and some undesirable, can be catalogued conveniently with respect to the Froude number.

The form shown in Figure 9A can be expected when the Froude number ranges from 1.7 to 2.5. When the Froude number is unity, the water would be flowing at critical depth; thus, a jump could not form. This would correspond to Point 0 on the specific energy diagram of Figure 4. For the values of Froude number between 1.0 and 1.7, there is only a slight difference in the conjugate depths D_1 and D_2. A slight ruffle on the water surface is the only apparent feature that differentiates this from flow with uniform velocity distribution. As the Froude number approaches 1.7, a series of small rollers develop on the surface as indicated in Figure 9A, and this action remains much the same but with further intensification up to a value of about 2.5. Actually, there is no particular stilling basin problem involved; the water surface is quite smooth, the velocity throughout is fairly uniform, and the energy loss is low.

Figure 9B indicates the type of jump that may be encountered at values of the Froude number from 2.5 to 4.5. This is an oscillating type of action, so common in canal structures, where the entering jet oscillates from bottom to surface and back again with no regular period. Turbulence occurs near the bottom one instant and entirely on the surface the next. Each oscillation produces a large wave of irregular period which, in the case of canals, can travel for miles doing unlimited damage to earth banks and riprap. The case is of sufficient importance that a separate section has been devoted to the practical aspects of design.

A well-stabilized jump can be expected for the range of Froude numbers between 4.5 and 9 (Figure 9C). In this range, the downstream extremity of the surface roller and the point at which the high-velocity jet tends to leave the floor practically occur in the same vertical plane. The jump is well balanced, and the action is thus at its best. The energy absorption in the jump for Froude numbers from 4.5 to 9 ranges from 45 to 70 percent (Figure 8).

As the Froude number increases above 9, the form of the jump gradually changes to that shown in Figure 9D. This is the case where V_1 is very high, D_1 is comparatively small, and the difference in conjugate depths is large. The high-velocity jet no longer carries through for the full length of the jump. In other words, the downstream extremity of the surface roller now becomes the determining factor in judging the length of the jump. Slugs of water rolling down the front face of the jump intermittently fall into the high-velocity jet, generating additional waves downstream, A rough surface can prevail. Figure 8 shows that the energy dissipation for this case may reach 85 percent.

The limits of the Froude number given above for the various forms of jump are not definite values, but overlap somewhat, depending on local factors. Returning to Figure 7, it is found that the length curve catalogs the various phases of the jump quite well. The flat portion of the curve indicates the range of the best operation. The steep portion of the curve to the left definitely indicates an internal change in the form of the jump. In fact, two changes are manifest, the form shown in Figure 9A and the form, which might better be called a transition stage, shown in Figure 9B. The right end of the curve on Figure 7 also indicates a change in form, but to less extent.

Practical Considerations

As stated previously, it was the intention to stress the academic rather than the practical viewpoint in this section. An exception has been made, as this is the logical place to point out a few of the practical aspects of stilling basin design using horizontal aprons. Viewing the four forms of jump just discussed, the following is pertinent:

1. All forms are encountered in stilling-basin design.

2. The form in Figure 9A requires no baffles or special consideration. The only requirement necessary is to provide the proper length of pool, which is relatively short. This can be obtained from Figure 7.

3. The form in Figure 9B is one of the most difficult to handle and is frequently encountered in the design of canal structures, diversion dams, and even outlet works. Baffle blocks or appurtenances in the basin are of little value. Waves are the main source of difficulty, and methods for coping with them are discussed. The present information may prove valuable in that it will help to restrict the use of jumps in the 2.5 to 4.5 Froude number range. In many cases its use cannot be avoided, but in other cases, altering of dimensions may bring the jump into the desirable range.

4. No particular difficulty is encountered in the form shown on Figure 9C. Arrangements of baffles and sills will be found valuable as a means of shortening the length of basin.

5. As the Froude number increases, the jump becomes more sensitive to tail-water depth. For numbers as low as 8, a tail-water depth greater than the conjugate depth is advisable, to be certain that the jump will stay on the apron. This phase will be discussed in more detail.

Conclusions

The foregoing experiments and discussion serve to associate the Froude number with stilling-basin design where it offers many advantages. The ratio of conjugate depths, the length of jump, the type of jump to be expected, and the losses involved have all been related to this number. The principal advantage of this form of presentation is that one may see the entire picture at a glance. The foregoing information is basic to the understanding of the hydraulic jump. The following sections deal with the more practical aspects, such as modifying the jump by baffles and sills to increase stability and shorten the length.

An example follows which may help clarify the information so far presented.

Water flowing under a sluice gate discharges into a rectangular stilling basin the same width as the gate. The average velocity and the depth of flow after contraction of the jet is complete are $V_1 = 85$ ft/sec and $D_1 = 5.6$ feet. Determine the conjugate tail-water depth, the length of basin required to confine the jump, the effectiveness of the basin to dissipate energy, and the type of jump to be expected.

$$F_1 = \frac{V_1}{\sqrt{gD_1}} = \frac{85}{\sqrt{32.2 \times 5.6}} = 6.34$$

Entering Figure 5 with this value

$$\frac{D_2}{D_1} = 8.5$$

The conjugate tail-water depth

$$D_2 = 8.5 \times 5.6 = 47.6 \text{ feet}$$

Entering the recommended curve on Figure 7 with a Froude number of 6.34

$$\frac{L}{D_2} = 6.13$$

Length of basin necessary to confine the jump

$$L = 6.13 \times 47.6 = 292 \text{ feet}$$

Entering Figure 8 with the above value of the Froude number, it is found that the energy absorbed in the jump is 58 percent of the energy entering.

By consulting Figure 9, it is apparent that a very satisfactory jump can be expected.

SHORT STILLING BASIN FOR CANAL STRUCTURES, SMALL OUTLET WORKS, AND SMALL SPILLWAYS (Basin III of Frontispiece)

Basin II, as shown in the frontispiece, often is considered too conservative and consequently overcostly for structures carrying small discharges at moderate velocities. This can be especially true in the case of canal chutes, drops, wasteways, and other structures which are constructed by the dozen on canal systems. Any saving that can be effected in decreasing the size of these structures can amount to a sizable sum when multiplied by the number of structures involved. In this section a generalized design is developed for a class of smaller structures in which the velocity at the entrance to the basin is moderate or low (5 to 60 feet per second, corresponding to an over-all head of about 100 feet). Further economies in basin length are accomplished with baffle piers.

Development

The most effective way to shorten a stilling basin is to modify the jump by the addition of appurtenances in the basin. One restriction imposed on these appurtenances, however, is that they must be self-cleaning or nonclogging. This restriction thus limits the appurtenances to blocks or sills which can be incorporated on the stilling-basin apron.

The Department of Agriculture developed a very short stilling basin designated "The SAF Basin," for use on drainage structures such as the Soil Conservation Service constructs. The SAF basin, Figure 16, fits the needs for which it was developed but does not provide the factor of safety necessary for Bureau use. This was demonstrated by constructing and testing several basins proportioned to SAF specifications. It was discovered, however, that the arrangement of this basin had excellent possibilities, and that by changing dimensions, such as the length, the tail-water depth, the height and location of the baffle blocks, etc., the desired degree of conservatism could be obtained.

In addition to the foregoing tests, numerous experiments were performed using various types and arrangements of baffle blocks on the apron in an effort to obtain the best possible solution. Some of the baffle blocks tried are shown on Figure 17. The blocks were positioned in both single and double rows with the second row staggered with respect to the first. Arrangement "a" on Figure 17 consisted of a solid bucket sill which was tried in several positions on the apron. This sill required an excessive tail-water depth to be effective. The solid sill was then replaced with blocks and spaces. For certain heights, widths, and spacing, block "b" performed quite well, resulting in a water surface similar to that shown on Figure 20. Block "c" was ineffective for any height. The velocity passed over the block at about 45° angle, thus was not impeded, and the water surface downstream was very turbulent with waves. The stepped block "d" was also ineffective both for a single row and a double row. The action was much the same as for "c". The cube "e" was effective when the best height, width, spacing, and position on the apron were found. The front of the jump was almost vertical and the water surface downstream was quite flat and smooth, much like the water surface shown on Figure 20. Block "f", which is the same shape used in the SAF basin, performed identically with the cubical block "e". The important feature as to shape appeared to be the vertical upstream face. The foregoing blocks were arranged in single and double rows. The second row in each case was of little value, sketch "h", Figure 17.

Block "g" is the same as block "f" with the corners rounded. It was found that rounding the corners greatly reduced the effectiveness of the blocks. In fact, a double row of blocks with rounded corners did not perform as well as a single row of blocks "b", "e", or "f". As block "f" is usually preferable from a construction standpoint, it was used throughout the remaining tests to determine a general design with respect to height, width, spacing, and position on the apron.

In addition to experimenting with the baffle blocks, variations were tried with respect to the size and shape of the chute blocks and the end sill. It was found that the chute blocks should be kept small, no larger than D_1, if possible. The end sill had little or no effect on the jump proper when baffle piers were placed as recommended. The basin as finally developed is shown on Figure 18. This basin is principally an impact dissipation device whereby the baffle blocks are called upon to do most of the work. The chute blocks aid in stabilization of the jump, and the solid-type end sill is for scour control.

Verification Tests

At the conclusion of the development work, a set of verification tests was made to examine and record the performance of this basin, which will be designated as Basin III, over the entire range of operating conditions that may be met in practice. The conditions under which the tests were run, the dimensions of the basin, and the results are recorded on Table 4. The symbols can be identified from Figure 18.

Stilling Basin Performance and Design

Stilling basin action was quite stable for this design; in fact, more so than for either Basins I or II. The front of the jump was steep, and there was less wave action to contend with downstream than in either of the former basins. In addition, Basin III has a large factor of safety against jump sweepout and operates equally well for all values of the Froude number above 4.5. The verification tests served to show that Basin III was very satisfactory.

Basin III should not be used where baffle piers will be exposed to velocities above the 50 to 60 feet per second range without the full realization that cavitation and resulting damage may occur. For velocities above 50 feet per second, Basin II or hydraulic model studies are recommended.

Table 4

VERIFICATION TESTS ON TYPE III STILLING BASIN

Flume	Test	Q Total discharge cfs	W	q Discharge per foot of Basin width	TW Tailwater depth ft	V_1 Velocity entering jump ft/sec	D_1 Depth entering jump ft	D_2/D_1	$F_1 = \frac{V_1}{\sqrt{gD_1}}$	L_{III} Length of jump ft	$\frac{L_{III}}{D_2}$	T_{so} Tailwater at sweep out ft	$\frac{T_{so}}{D_1}$	$\frac{T_{so}}{D_2}$	θ Slope of Chute
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
B	1	2,500	2.000	1.250	1.120	17.36	0.072	15.56	11.41	2.90	2.59	0.94	13.65	0.84	0.7:1
	2	4.000		2.000	1.430	17.54	0.114	12.54	9.16	3.70	2.59	1.11	9.73	0.78	
	3	6.000		3.000	1.750	17.65	0.170	10.29	7.54	4.50	2.57	1.29	7.58	0.74	
	4	8.000		4.000	2.030	17.86	0.224	9.06	6.64	4.90	2.41	1.57	7.00	0.77	
C	5	1.600	1.500	1.067	1.070	17.49	0.061	17.54	12.48	3.00	2.80	0.88	14.42	0.82	2:1
	6	2.630		1.753	1.350	18.26	0.096	14.06	10.39	3.80	2.81	1.16	12.08	0.86	
	7	2.750		1.833	1.400	18.33	0.100	14.00	10.21	4.20	3.00	1.17	11.70	0.84	
	8	4.000		2.667	1.785	20.36	0.131	13.62	9.91	5.00	2.80	1.42	10.84	0.80	
D	9	5.000	3.970	1.259	1.250	20.30	0.062	20.16	14.38	3.20	2.56	1.04	16.77	0.83	0.6:1
	10	6.000		1.511	1.350	20.41	0.074	18.24	13.21	3.70	2.74	1.12	15.13	0.83	
	11	11.00		2.771	1.860	21.15	0.131	14.20	10.29	5.00	2.69	1.50	11.45	0.81	
	12	13.00		3.274	2.020	21.40	0.153	13.20	9.64	5.20	2.57	1.65	10.78	0.82	
	13	20.00		5.038	2.585	23.00	0.219	11.80	8.66	6.46	2.50	2.15	9.82	0.83	
E	14	5.000	3.970	1.259	0.840	10.49	0.120	7.00	5.33	2.10	2.50	0.70	5.83	0.83	Varied

Flume	Test	h_1 Height of chute blocks ft	$\frac{h_1}{D_1}$	$\frac{w_1}{h_1}$	$\frac{s_1}{h_1}$	h_3 Height of baffle blocks ft	$\frac{h_3}{D_1}$	$\frac{w_3}{h_3}$	$\frac{s_3}{h_3}$	Distance to baffles ft	$\frac{L_{III}}{3D_2}$	h_4 Height of end sill ft	$\frac{h_4}{D_1}$	Z Depth upstream from baffles ft	$\frac{Z}{D_2}$
		(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)
B	1	0.073	1.01	1.0	1.0	0.167	2.32	1.0	1.0	0.800	0.714	0.125	1.74	0.60	0.54
	2	0.114	1.00	1.0	1.0	0.218	1.91	1.0	1.0	0.920	0.643	0.187	1.64	0.80	0.56
	3	0.333	1.96	0.6	0.6	0.302	1.78	1.0	1.0	1.200	0.686	0.250	1.47	0.95	0.54
	4	0.229	1.02	1.0	1.0	0.396	1.77	1.0	1.0	1.340	0.660	0.302	1.35	1.20	0.59
C	5	0.062	1.02	1.0	1.0	0.167	2.74	0.75	0.75	0.850	0.794	0.092	1.51	0.60	0.56
	6	0.100	1.04	1.0	1.0	0.240	2.50	0.75	0.75	1.000	0.741	0.146	1.52	0.65	0.48
	7	0.146	1.46	1.0	1.0	0.250	2.50	0.75	0.75	1.210	0.864	0.156	1.56	0.70	0.50
	8	0.187	1.43	0.75	0.75	0.312	2.38	0.75	0.75	1.430	0.801	0.219	1.67	0.90	0.50
D	9	0.062	1.00	1.0	1.0	0.188	3.03	1.0	1.0	1.000	0.800	0.125	2.02	0.60	0.48
	10	0.083	1.12	1.0	1.0	0.208	2.81	1.0	1.0	1.120	0.830	0.135	1.82	0.65	0.48
	11	0.135	1.03	1.0	1.0	0.302	2.31	1.0	1.0	1.250	0.672	0.208	1.59	0.95	0.51
	12	0.156	1.02	1.0	1.0	0.354	2.31	1.0	1.0	1.680	0.832	0.208	1.36	1.05	0.52
	13	0.219	1.00	1.0	1.0	0.479	2.19	0.75	0.75	2.153	0.833	0.271	1.24	1.30	0.50
E	14	0.122	1.02	1.0	1.0	0.215	1.79	0.75	0.75	0.672	0.833	0.150	1.25	0.55	0.65

Chute Blocks

The recommended proportions for Basin III are shown on Figure 18. The height, width, and spacing of the chute blocks are equal to D_1. Larger heights were tried, as can be observed from Column 18, Table 4, but are not recommended. The larger chute blocks tend to throw a portion of the high-velocity jet over the baffle blocks. Some cases will be encountered in design, however, where D_1 is less than 8 inches. In such cases, the blocks may be made 8 inches high, which is considered by some designers to be the minimum size possible from a construction standpoint. The width and spacing are the same as the height, but this may be varied so long as the aggregate width of spaces approximately equals the total width of the blocks.

Baffle Blocks

The height of the baffle blocks increases with the Froude number, as can be observed from Columns 22 and 10, Table 4. The height, in terms of D_1, can be obtained from the upper line on Figure 19. The width and spacing can vary so long as the total spacing is equal to the total width of blocks. The most satisfactory width and spacing were found to be three-fourths of the height. It is not necessary to stagger the baffle blocks with the chute blocks, as this is often difficult and there is little to be gained from a hydraulic standpoint.

The baffle blocks are located $0.8D_2$ downstream from the chute blocks, as shown in Figure 18. The actual positions used in the verification tests are shown in Column 25, Table 4. The position, height and spacing of the baffle blocks on the apron should be adhered to carefully, as these dimensions are important. For example, if the blocks are set appreciably upstream from the position shown, they will produce a cascade with resulting wave action. On the contrary, if the blocks are set farther downstream than shown, a longer basin will be required. Likewise, if the baffle blocks are too high, they can produce a cascade, while if too low a rough water surface will result. It is not the intention to give the impression that the position or height of the baffle blocks are critical. Their position or height are not critical so long as the above proportions are followed. There exists a reasonable amount of leeway in all directions; however, one cannot place the baffle blocks on the pool floor at random and expect anything like the excellent action associated with the Type III basin.

The baffle blocks may be in the form shown on Figure 18, or they may be cubes; either shape is effective. The corners of the baffle blocks are not rounded, as the sharp edges are effective in producing eddies, which in turn aid in the dissipation of energy. It is advisable to place reinforcing steel back at least 6 inches from the block surfaces when possible, as there is some evidence that steel placed close to the surface aids spalling.

End Sill

The height of the solid end sill is also shown to vary with the Froude number, although there is nothing critical about this dimension. The heights of the sills used in the verification tests are shown in Columns 27 and 28 of Table 4. The height of the end sill in terms of D_1 is plotted with respect to the Froude number and shown as the lower line on Figure 19. A slope of 2:1 was used throughout the tests.

Tail-Water Depth

The SAF rules suggest the use of a tail-water depth less than full conjugate depth, D_2. Full conjugate depth, measured above the apron, is recommended for Basin III. There are several reasons for this statement: First, the best operation for this stilling basin occurs at full conjugate tail-water depth; secondly, if less than the conjugate depth is used, the surface velocities leaving the pool are high, the jump action is inpaired, and there is a greater chance for scour downstream; and thirdly, if the baffle blocks erode with time, the additional tail-water depth will serve to lengthen the interval between repairs. On the other hand, there is no particular advantage to using greater than the conjugate depth, as the action in the pool will show little or no improvement.

The margin of safety for Basin III varies from 15 to 18 percent depending on the value of the Froude number, as can be observed by the dotted line labeled, "Minimum Tail Water Depth--Basin III", on Figure 11. The points, from which the line was drawn, were obtained from the verification, tests, Columns 10 and 14, Table 4. Again, this line does not represent complete sweepout, but the point at which the front of the jump moves away from the chute blocks and the basin no longer functions properly. In special cases, it may be necessary to encroach on this wide margin of safety; however, it is not advisable as a general rule, for the reasons stated above.

Length of Basin

The length of Basin III, which is related to the Froude number, can be obtained by consulting the lower curve on Figure 12. The points, indicated by circles, were obtained from Columns 10 and 12, Table 4, and indicate the extent of the verification tests. The length is measured from the downstream side of the chute blocks to the downstream edge of the end sill, Figure 18. Although this curve was determined conservatively, it will be found that the length of Basin III is less than one-half the length needed for a basin without appurtenances. Basin III may be effective for values of the Froude number as low as 4.5; thus, the length curve was terminated at this value.

Water Surface and Pressure Profiles

Approximate water-surface profiles were obtained for Basin III during the verification tests. The front of the jump was so steep, Figure 20, that only two measurements were necessary - the tail-water depth and the depth upstream from the baffle blocks. The tail-water depth is shown in Column 6, and the upstream depth is recorded in Column 29 of Table 4. The ratio of the upstream depth to conjugate depth is shown in Column 30. As can be observed, the ratio is much the same regardless of the value of the Froude number. The average of the ratios in Column 30 is 0.52. Thus, it will be assumed that the depth upstream from the baffle blocks is one-half the tail-water depth.

The profile represented by the crosshatched area, Figure 20, is for conjugate tail-water depth. For a greater tail-water depth, D_z the upstream depth would be $\frac{D_z}{2}$. For a tail-water depth less than conjugate, D_y, the upstream depth would be approximately $\frac{D_y}{2}$. There appears to be no particular significance to the fact that this ratio is one-half.

The information on Figure 20 applies only to Basin III, proportioned according to the rules set forth. It can be assumed that for all practical purposes the pressure and water-surface profiles are the same. There will be a localized increase in pressure on the apron immediately upstream from each baffle block, but this has been taken into account, more or less, by extending the diagram to full tail-water depth, beginning at the upstream face of the baffle blocks.

Recommendations

The following rules pertain to the design of the Type III basin, Figure 18:

1. The stilling basin operates best at full conjugate tail-water depth, D_2. A reasonable factor of safety is involved at conjugate depth for all values of the Froude number (Figure 11), but it is recommended that the designer not make a general practice of encroaching on this margin of safety.

2. The length of pool, which is less than one-half the length of the natural jump, can be obtained by consulting the curve for Basin III on Figure 12.

3. Stilling Basin III may be effective for values of the Froude number as low as 4.0, but this cannot be stated for certain.

4. Height, width, and spacing of chute blocks should equal the average depth of flow entering the basin, or D_1. Width of blocks may be decreased, providing spacing is reduced a like amount. Should D_1 prove to be less than 8 inches, make the blocks 8 inches high.

5. The height of the baffle blocks varies with the Froude number and is given on Figure 19. The blocks may be cubes or they may be constructed as shown on Figure 18 so long as the upstream face is vertical and in one plane. This feature is important. The width and spacing of baffle blocks are also shown on Figure 18. In narrow structures where the specified width and spacing of blocks do not appear practical, block width and spacing may be reduced, providing they are reduced a like amount. A half space is recommended adjacent to the walls.

6. The upstream face of the baffle blocks should be set at a distance of $0.8D_2$ from the downstream face of the chute blocks (Figure 18). This dimension is also important.

7. The height of the solid sill at the end of the basin can be obtained from Figure 19. The slope is 2:1 upward in the direction of flow.

8. There is no need to round or streamline the edges of the chute blocks, end sill or baffle piers. Streamlining of baffle piers may result in loss of half of their effectiveness. Small chamfers to prevent chipping of the edges is permissable.

9. As a reminder, a condition of excess tail-water depth does not justify shortening the basin length.

10. It is recommended that a radius of reasonable length ($R \geqq 4D_1$) be used at the intersection of the chute and basin apron for slopes of 45° or greater.

11. As a general rule the slope of the chute has little effect on the jump unless long flat slopes are involved. This phase will be considered in Section 5 on sloping aprons.

As the Type III basin is short coupled, the above rules should be followed closely for its proportioning. If the proportioning is to be varied from that recommended, or if the limits given below are exceeded, a model study is advisable. Arbitrary limits for the Type III basin are set at 200 cfs per foot of basin width, or 100 feet of fall, until experience demonstrates otherwise.

Example

Given the following computed values for a small overflow dam:

Q cfs	q cfs	V_1 ft/sec	D_1 ft
3,900	78.0	69	1.130
3,090	61.8	66	0.936
2,022	40.45	63	0.642
662	13.25	51	0.260

and the tail-water curve for the river, identified by the solid line on Figure 21, proportion a Type III basin for the most adverse condition, utilizing full conjugate tail-water depth. The flow is symmetrical and the width of the basin is 50 feet. (The purpose of this example is to demonstrate the use of the jump height curve.)

The first step is to compute the jump height curve. As V_1 and D_1 are given, the Froude number is computed and tabulated in Column 2, Table 5, below:

Table 5

Q cfs (1)	F_1 (2)	$\frac{D_2}{D_1}$ (3)	D_1 ft (4)	D_2 ft (5)	Jump height elevation Curve a (6)	Curve a' (7)
3,900	11.42	15.75	1.130	17.80	617.5	615.0
3,090	12.02	16.60	0.936	15.54	615.2	612.7
2,022	13.85	19.20	0.642	12.33	612.0	609.5
662	17.62	24.50	0.260	6.37	606.1	603.6

Entering Figure 11 with these values of the Froude number, values of $\frac{TW}{D_1}$ are obtained for conjugate tail-water depth from the solid line. These values are also $\frac{D_2}{D_1}$ and are shown listed in Column 3. The conjugate tail-water depths for the various discharges, Column 5, were obtained by multiplying the values in Column 3 by those in Column 4.

If it were assumed that the most adverse operating condition occurs at the maximum discharge of 3,900 cfs, the stilling basin apron should be placed at elevation 617.5 - 17.8 or elevation 599.7.

With the apron at elevation 599.7 the tail-water required for conjugate tail-water depth for each discharge would follow the elevations listed in Column 6. Plotting Columns 1 and 6 on Figure 21 results in Curve a, which shows that the tail-water depth is inadequate for all but the maximum discharge.

The tail-water curve is unusual in that the most adverse tail-water condition occurs at a discharge of approximately 2,850 cfs rather than maximum. As full conjugate tail-water depth is desired for the most adverse tail-water condition, it is necessary to shift the jump height curve downward to match the tail-water curve for a discharge of 2,850 cfs (see Curve a', Figure 21). The coordinates for Curve a' are given in Columns 1 and 7, Table 5. This will place the basin floor 2.5 feet lower, or elevation 597.2 feet, as shown in the sketch on Figure 21.

Although the position of the basin floor was set for a discharge of 2,850 cfs, the remaining details are proportioned for the maximum discharge 3,900 cfs.

Entering Figure 12 with a Froude number of 11.42,

$$\frac{L_{III}}{D_2} = 2.75, \text{ and the length of}$$

basin required $L_{III} = 2.75 \times 17.80 = \underline{48.95}$ feet.

(Notice that conjugate depth was used, not tail-water depth.)

The height, width, and spacing of chute blocks are equal to D_1 or 1.130 feet (use <u>13</u> or <u>14</u> inches).

The height of the baffle blocks for a Froude number of 11.42 (Figure 19) is $2.5D_1$.

$$h_3 = 2.5 \times 1.130 = 2.825 \text{ feet (use } \underline{34} \text{ inches).}$$

The width and spacing of the baffle blocks are preferably three-fourths of the height or

$$0.75 \times 34 = \underline{25.5} \text{ inches.}$$

From Figure 18, the upstream face of the baffle blocks should be $0.8D_2$ from the downstream face of the chute blocks, or

$$0.8 \times 17.80 = \underline{14.24} \text{ feet.}$$

The height of the solid end sill (Figure 19) is $1.60D_1$, or

$$h_4 = 1.60 \times 1.130 = 1.81 \text{ feet (use } \underline{22} \text{ inches).}$$

The final dimensions of the basin are shown on Figure 21.

STILLING BASIN DESIGN AND WAVE SUPPRESSORS FOR CANAL STRUCTURES, OUTLET WORKS AND DIVERSION DAMS
(Basin IV in Frontispiece)

In this section the characteristics of the hydraulic jump and the design of an adequate stilling basin for Froude numbers between 2.5 and 4.5 are discussed. This range is encountered principally in the design of canal structures, but occasionally diversion dams and outlet works fall in this category. In the 2.5 to 4.5 Froude number range, the jump is not fully developed and the previously discussed methods of design do not apply. The main problem concerns the waves created in the hydraulic jump, making the design of a suitable wave suppressor a part of the stilling basin problem.

Four means of reducing wave heights are discussed. The first is an integral part of the stilling basin design and should be used only in the 2.5 to 4.5 Froude number range. The second may be considered to be an alternate design and may be used over a greater range of Froude numbers. These types are discussed as a part of the stilling basin design. The third and fourth devices are considered as appurtenances which may be included in an original design or added to an existing structure. Also, they may be used in any open channel flow-way without consideration of the Froude number. These latter devices are described under the heading Wave Suppressors.

Jump Characteristics - Froude Numbers 2.5 to 4.5

For low values of the Froude number, 2.5 to 4.5, the entering jet oscillates intermittently from bottom to surface, as indicated in Figure 9B, with no particular period. Each oscillation generates a wave which is difficult to dampen. In narrow structures, such as canals, waves may persist to some degree for miles. As they encounter obstructions in the canal, such as bridge piers, turnouts, checks, and transitions, reflected waves may be generated which tend to dampen, modify, or intensify the original wave. Waves are destructive to earth-lined canals and riprap and produce undesirable surges at gaging stations and in measuring devices. Structures in this range of Froude numbers are the ones that require the most maintenance. In fact, it has been necessary to replace or rebuild a number of existing structures in this category.

On wide structures, such as diversion dams, wave action is not as pronounced since the waves can travel laterally as well as parallel to the direction of flow. The combined action produces some dampening effect but also results in a choppy water surface. These waves may or may not be dissipated in a short distance. Where outlet works, operating under heads of 50 feet or greater, fall within the range of Froude numbers between 2.5 and 4.5, a model study of the stilling basin is imperative. A model study is the only means of including preventive or corrective devices in the structure so that proper performance can be assured.

Stilling Basin Design - Froude Numbers 2.5 to 4.5

Development Tests

The best way to combat a wave problem is to eliminate the wave at its source; in other words, concentrate on altering the condition which generates the wave. In the case of the stilling basin preceded by an overfall or chute, two schemes were apparent for eliminating waves at their source. The first was to break up or eliminate the entering jet, shown on Figure 9B, by opposing it with directional jets deflected from baffle piers or sills. The second was to bolster or intensify the roller, shown in the upper portion of Figure 9B, by directional jets deflected from large chute blocks.

The first method was unsuccessful in that the number and size of appurtenances necessary to break up the roller occupied so much volume that these in themselves posed an obstruction to the flow. This conclusion was based on tests in which various shaped baffle blocks and guide blocks were systematically placed in a stilling basin in combination with numerous types of spreader teeth and deflectors in the chute. The program involved dozens of tests, and not until all conceivable ideas were tried was this approach abandoned. A few of the basic ideas tested are shown on Figure 22, a, b, c, f, g, and h.

Final Tests

Deflector blocks. The second approach, that of attempting to intensify the roller, yielded better results. In this case, large blocks were placed well up on the chute, while nothing was installed in the stilling basin proper. The object in this case was to direct a jet at the base of the roller in an attempt to strengthen it. After a number of trials, the roller was actually intensified, which did improve the stability of the jump. Sketches d and e on Figure 22 indicate the only schemes that showed promise, although many variations were tried. After finding an arrangement that was effective, it was then attempted to make the field construction as simple as possible. The dimensions and proportions of the deflector blocks as finally adopted are shown on Figure 23.

The object in the latter scheme was to place as few appurtenances as possible in the path of the flow, as volume occupied by appurtenances helps to create a backwater problem, thus requiring higher training walls. The number of deflector blocks shown on Figure 23 is a minimum requirement to accomplish the purpose set forth. The width of the blocks is shown equal to D_1 and this is the maximum width recommended. From a hydraulic standpoint, it is desirable that the blocks be constructed narrower than indicated, preferably $0.75D_1$. The ratio of block width to spacing should be maintained as 1:2.5. The extreme tops of the blocks are $2D_1$ above the floor of the stilling basin. The blocks may appear to be rather high and, in some cases, extremely long, but this is essential as the jet must play at the base of the roller to be effective. To accommodate the various slopes of chutes and ogee shapes encountered, a rule has been established that the horizontal length of the blocks should be at least $2D_1$. The upper surface of each block is sloped at 5° in a downstream direction, as it was found that this feature resulted in better operation, especially at the lower discharges.

Tail-water depth. A tail-water depth 5 to 10 percent greater than the conjugate depth is strongly recommended for the above basin. Since the jump is very sensitive to tail-water depth at these low values of the Froude number, a slight deficiency in tail-water depth may allow the jump to sweep completely out of the basin. Many of the difficulties that have been encountered in small field structures in the past can be attributed to this aspect of the jump for low numbers. In addition, the jump performs much better and wave action is diminished if the tail-water depth is increased to approximately $1.1D_2$.

Basin length and end sill. The length of this basin, which is relatively short, can be obtained from the upper curve on Figure 12. No additional blocks or appurtenances are needed in the basin, as these will prove a greater detriment than aid. The addition of a small triangular sill placed at the end of the apron for scour control is desirable.

Performance. If designed for the maximum discharge, this stilling basin will perform satisfactorily for all flows. Waves below the stilling basin will still be in evidence but will be of the ordinary variety usually encountered with jumps of a higher Froude number. This design is applicable to rectangular cross sections only.

Alternate Stilling Basin Design - Small Drops

Performance

An alternate basin for reducing wave action at the source, for values of the Froude number between 2.5 and 4.5, is applicable to small drops in canals. The Froude number in this case would be computed the same as though the drop were an overflow crest. A series of steel rails, channel irons or timbers in the form of a grizzly are installed at the drop, as shown on Figure 24.

The overfalling jet is separated into a number of long, thin sheets of water which fall nearly vertically into the canal below. Energy dissipation is excellent and the usual wave problem is avoided. If the rails are tilted downward at an angle of 3^o or more, the grid is self-cleaning.

Design

Two spacing arrangements were tested in the laboratory: in the first, the spacing was equal to the width of the beams, and in the second, the spacing was two-thirds of the beam width. The latter was the more effective. In the first, the length of grizzly required was about 2.9 times the depth of flow (y) in the canal upstream, while in the second, it was necessary to increase the length to approximately 3.6y. The following expression can be used for computing the length of grizzly:

$$LG = \frac{Q}{CWN\sqrt{2gy}} \qquad (4)$$

Where Q is total discharge, C is an experimental coefficient, W is the width of spacing in feet, N is the number of spaces, g is the acceleration of gravity and y is the depth of flow in the canal upstream (see Figure 24). The value of C for the two arrangements tested was 0.245.

In this case, the grizzly makes it possible to avoid the hydraulic jump. Should it be desired to maintain a certain level in the canal upstream, the grid may be tilted upward to act as a check; however, this arrangement may pose a cleaning problem.

Wave suppressors

The two stilling basins described above may be considered to be wave suppressors, although the suppressor effect is obtained from the necessary features of the stilling basin. If greater wave reduction is required on a proposed structure, or if a wave suppressor is required to be added to an existing flow-way, the two types discussed below may prove useful. Both of these types are applicable to most open channel flow-ways having rectangular, trapezoidal, or other cross-sectional shapes. The first, or raft type, may prove more economical than the second, or underpass type, but rafts may not provide the degree of wave reduction obtainable with the underpass type. Both types may be used without regard to the Froude number.

Raft Type Wave Suppressor

In a structure of the type shown in Figure 25, there are no means for eliminating waves at their source. Tests showed that appurtenances in the stilling basin merely produced severe splashing and created a backwater effect, resulting in submerged flow at the gate for the larger flows. Submerged flow reduced the effective head on the structure, and in turn, the capacity. Tests on several suggested devices showed that rafts provided the best answer to the wave problem when additional submergence could not be tolerated. The general arrangement of the tested structure is shown in Figure 25. The Froude number varied from 3 to 7, depending on the head behind the gate and the gate opening. Velocities in the canal ranged from 5 to 10 feet per second. Waves were 1.5 feet high, measured from trough to crest.

During the course of the experiments, a number of rafts were tested; thick rafts with longitudinal slots, thin rafts made of perforated steel plate, and others, both floating and fixed. Rigid and articulated rafts were tested in various arrangements.

The most effective raft arrangement consisted of two rigid stationary rafts 20 feet long by 8 feet wide, made from 6- by 8-inch timbers, placed in the canal downstream from the stilling basin (Figure 25). A space was left between timbers, and lighter cross pieces were placed on the rafts parallel to the flow, giving the appearance of many rectangular holes. Several essential requirements for the raft were apparent: (1) that the rafts be perforated in a regular pattern; (2) that there be some depth to these holes; (3) that at least two rafts be used; and (4) that the rafts be rigid and held stationary.

It was found that the ratio of hole area to total area of raft could be from 1:6 to 1:8. The 8-foot width, W on Figure 25, is a minimum dimension. The rafts must have sufficient thickness so that the troughs of the waves do not break free from the underside. The top surfaces of the rafts are set at the mean water surface in a fixed position, so that they cannot move. Spacing between rafts should be at least three times the raft dimension, measured parallel to the flow. The first raft decreases the wave height about 50 percent, while the second raft effects a further reduction. Surges over the raft dissipate themselves by flow downward through the holes. For this specific case, the waves were reduced from 18 to 3 inches in height.

Under certain conditions wave action is of concern only at the maximum discharge when freeboard is endangered, so the rafts can be a permanent installation. Should it be desired to suppress the waves at partial flows, the rafts may be made adjustable, or, in the case of trapezoidal channels, a second set of rafts may be placed under the first set for partial flows. The rafts should perform equally well in trapezoidal as well as rectangular channels.

The recommended raft arrangement is also applicable for suppressing waves with a regular period such as wind waves, waves produced by the starting and stopping of pumps, etc. In this case, the position of the downstream raft is important. The second raft should be positioned downstream at some fraction of the wave length. Placing it at a full wave length could cause both rafts to be ineffective. Thus, for narrow canals it may be advisable to make the second raft portable. However, if it becomes necessary to make the rafts adjustable or portable, or if a moderate increase in depth in the stilling basin can be tolerated, consideration should be given to the type of wave suppressor discussed below.

Underpass Type Wave Suppressor

By far the most effective wave dissipator is the short-tube type of underpass suppressor. The name "short-tube" is used because the structure has many of the characteristics of the short-tube discussed in hydraulics textbooks. This wave suppressor may be added to an existing structure or included in the original construction. In either case it provides a sightly structure, permanent in nature, which is economical to construct and effective in operation.

The recommendations for this structure are based on three separate model investigations, each having different flow conditions and wave reduction requirements.

Essentially, the structure consists of a horizontal roof placed in the flow channel with a headwall sufficiently high to cause all flow to pass beneath the roof. The height of the roof above the channel floor may be set to effectively reduce wave heights for a considerable range of flows or channel stages. The length of the roof, however, determines the amount of wave suppression obtained for any particular roof setting.

Performance

The effectiveness of this wave suppressor is illustrated in Figure 26. In this instance, it was desired to reduce wave heights entering a lined canal to prevent overtopping of the canal lining at near maximum discharges. Below 3,000 second-feet, waves were in evidence but did not overtop the lining. For larger discharges, however, the stilling basin produced moderate waves which were actually intensified by the short transition between the basin and the canal. These intensified waves overtopped the lining at 4,000 second-feet and became a real problem at 4,500 second-feet. Anxiety developed when it became known that water demands would soon require 5,000 second-feet, the design capacity of the canal. Tests were made with a suppressor 21 feet long using discharges from 2,000 to 5,000 second-feet. The suppressor was located between the stilling basin and the canal.

Figure 27, Test 1, shows the results of tests to determine the optimum opening between the roof and the channel floor, using the maximum discharge, 5,000 second-feet. With a 14-foot opening, waves were reduced from about 8 feet to about 3 feet. Waves were reduced to less than 2 feet with an opening of 11 feet. Smaller openings produced less wave height reductions, due to the turbulence created at the underpass exit. Thus, it may be seen that an opening of from 10 to 12 feet produced optimum results.

With the opening set at 11 feet, the suppressor effect was then determined for other discharges. These results are shown on Figure 27, Test 2. Wave height reduction was about 78 percent at 5,000 second-feet, increasing to about 84 percent at 2,000 second-feet. The device became ineffective at about 1,500 second-feet, when the depth of flow became less than the height of the roof.

To determine the effect of suppressor length on the wave reduction, other factors were held constant while the length was varied. Tests were made on suppressors 10, 21, 30, and 40 feet long for discharges of 2, 3, 4, and 5 thousand second-feet, Figure 27, Test 3. Roof lengths in terms of the downstream depth D_2 for 5,000 second-feet were $0.62D_2$, $1.31D_2$, and $2.5D_2$, respectively. In terms of a 20-foot-long underpass, halving the roof length almost doubled the downstream wave height while doubling the 20-foot length almost halved the resulting wave height.

The same type of wave suppressor was successfully used in an installation where it was necessary to obtain optimum wave height reductions, since flow from the underpass discharged directly into a Parshall flume, in which it was desired to obtain accurate discharge measurements. The capacity of the structure was 625 cubic feet per second, but it was necessary for the underpass to function for low flows as well as for the maximum. With an underpass $3.5D_2$ long and set as shown in Figure 28, the wave reductions were as shown in Table 6.

Table 6

WAVE HEIGHTS IN FEET--PROTOTYPE
Maximum Head

Discharge in cfs	625		550		400		200		100	
	Upstream*	Downstream*	U	D	U	D	U	D	U	D
Wave heights in feet	**3.8 plus	0.3	4.2	0.3	4.5	0.4	3.6	0.4	1.7	0.3

*Upstream station is at end of stilling basin. Downstream station is in Parshall flume.
**Recorder pen reached limit of travel in this test only.

Figure 28 shows some of the actual wave traces recorded by an oscillograph. Here it may be seen that the maximum wave height, measured from minimum trough to maximum crest did not occur on successive waves. Thus, the water surface will appear smoother to the eye than is indicated by the maximum wave heights recorded in Table 6.

General Design Procedure

To design an underpass for a particular structure there are three main considerations: First, how deeply should the roof be submerged; second, how long an underpass should be constructed to accomplish the necessary wave reduction; and third, how much increase in flow depth will occur upstream from the underpass. These considerations are discussed in order.

Based on the two installations shown on Figures 27 and 28, and on other experiments, it has been found that maximum wave reduction occurs when the roof is submerged about 33 percent, i.e., when the under side of the underpass is set 33 percent of the flow depth below the water surface for maximum discharge, Figure 29C. Submergences greater than 33 percent (for the cases tested) produced undesirable turbulence at the underpass outlet, resulting in less overall wave reduction. With the usual tail-water curve, submergence and the percent reduction in wave height will become less, in general, for smaller than maximum discharges. This is illustrated by the upper curve in Figure 29C. The lower curve shows a near constant value for less submergence, but it is felt that this is a somewhat special case since the wave heights for less than maximum discharge were smaller and of shorter period than in the usual case.

It is known that the wave period greatly affects the performance of a given underpass, with the greatest wave reduction occuring for short period waves. Since the designer usually does not know in advance the wave periods to be expected, this factor should be eliminated from design consideration as far as possible. Fortunately, wave action below a stilling basin usually has no measurable period but consists of a mixture of generated and reflected waves best described as a choppy water surface. This fact makes it possible to provide a practical solution from limited data and to eliminate the wave period from consideration, except in this general way: waves must be of the variety ordinarily found downstream from hydraulic jumps or energy dissipators. These usually have a period of not more than about 5 seconds. Longer period waves may require special treatment not covered in this discussion. Fortunately, too, there generally is a tendency for the wave period to become less with decreasing discharge. Since the suppressor provides a greater percentage reduction on shorter period waves, this tends to offset the characteristics of the device to give less wave reduction for reduced submergence at lower discharges. It is, therefore, advisable in the usual case to submerge the underpass about 33 percent for the maximum discharge. For less submergence, the wave reduction can be estimated from Figure 29C.

The minimum length of underpass required depends on the amount of wave reduction considered necessary. If it is sufficient to obtain a nominal reduction to prevent overtopping of a canal lining at near maximum discharge or to prevent waves from attacking channel banks, a length $1D_2$ to $1.5D_2$ will provide from 60 to 75 percent wave height reduction, provided the initial waves have periods up to about

5 seconds. The shorter the wave period the greater the reduction for a given underpass. For long period waves, little wave reduction may occur because of the possibility of the wave length being nearly as long or longer than the underpass, with the wave passing untouched beneath the underpass.

To obtain greater than 75 percent wave reductions, a longer underpass is necessary. Under ideal conditions, an underpass $2D_2$ to $2.5D_2$ in length may provide up to 88 percent wave reduction for wave periods up to about 5 seconds. Ideal conditions include a velocity beneath the underpass of less than, say, 10 feet per second and a length of channel 3 to 4 times the length of the underpass downstream from the underpass which may be used as a quieting pool to still the small turbulence waves created at the underpass exit.

Wave height reduction up to about 93 percent may be obtained by using an underpass $3.5D_2$ to $4D_2$ long. Included in this length is a 4:1 sloping roof extending from the underpass roof elevation to the tail-water surface. The sloping portion should not exceed about one-quarter of the total length of underpass. Since slopes greater than 4:1 do not provide the desired draft-tube action, they should not be used. Slopes flatter than 4:1 provide better draft-tube action and are, therefore, desirable.

Since the greatest wave reduction occurs in the first D_2 of underpass length, it may appear advantageous to construct two short underpasses rather than one long one. In the one case tested, two underpasses each $1D_2$ long, with a length $5D_2$ between them, gave an added 10 percent wave reduction advantage over one underpass $2D_2$ long. The extra cost of another headwall should be considered, however.

Table 7 summarizes the amount of wave reduction obtainable for various underpass lengths.

Table 7

EFFECT OF UNDERPASS LENGTH ON WAVE REDUCTION
For Underpass Submergence 33 Percent and
Maximum Velocity Beneath 14 ft/sec

Underpass length	Percent wave reduction*
$1D_2$ to $1.5D_2$	60 to 75
$2D_2$ to $2.5D_2$	80 to 88
3.5 to $4.0D_2$	**90 to 93

*For wave periods up to 5 seconds.
**Upper limit only with draft-tube type outlet.

To determine the backwater effect of placing the underpass in the channel, Figure 29B will prove helpful. Data from four different underpasses were used to obtain the two curves shown. Although the test points from which the curves were drawn showed minor inconsistencies, probably because factors other than those considered also affected the depth of water upstream from the underpass, it is believed that the submitted curves are sufficiently accurate for design purposes. Figure 29B shows two curves of the discharge coefficient "C" versus average velocity beneath the underpass, one for underpass lengths of $1D_2$ to $2D_2$ and the other for lengths $3D_2$ to $4D_2$. Intermediate values may be interpolated, although accuracy of this order is not usually required.

Example

To illustrate the use of the preceding data in designing an underpass, a sample problem will be helpful.

Assume a rectangular channel 30 feet wide and 14 feet deep flows 10 feet deep at maximum discharge, 2,400 cfs. It is estimated that waves will be 5 feet high and of the ordinary variety having a period less than 5 seconds. It is desired to reduce the height of the waves to approximately 1 foot at maximum discharge by installing an underpass-type wave suppressor without increasing the depth of water upstream from the underpass more than 15 inches.

To obtain maximum wave reduction at maximum discharge, the underpass should be submerged 33 percent. Therefore, the depth beneath the underpass is 6.67 feet with a corresponding velocity of 12 ft/sec, $(V = \frac{2,400}{30 \times 6.67})$. To reduce the height of the waves from 5 to 1 foot, an 80 percent reduction in wave heights is indicated, and, from Table 7, requires an underpass approximately $2D_2$ in length.

From Figure 29B, C = 1.07 for $2D_2$ and a velocity of 12 ft/sec.

From the equation given on Figure 29B:

$$\text{Total head, } h + h_v = \left(\frac{Q}{CA\sqrt{2g}}\right)^2 = \left(\frac{2,400}{8.02 \times 1.07 \times 200}\right)^2 = 1.95 \text{ feet}$$

$h + h_v$ is the total head required to pass the flow and h represents the backwater effect or increase in depth of water upstream from the underpass. The determination of values for h and h_v is done by trial and error. As a first determination, assume that $h + h_v$ represents the increase in head.

Then channel approach velocity, $V_1 = \frac{Q}{A} =$

$$\frac{2,400}{(10 + 1.95)30} = 6.7 \text{ ft/sec}$$

$$h_v = \frac{(V_1)^2}{2g} = \frac{(6.7)^2}{64.4} = 0.70 \text{ foot}$$

and h = 1.95 - 0.70 = 1.25 feet

To refine the calculation, the above computation is repeated using the new head

$$V_1 = \frac{2,400}{(10 + 1.25)30} = 7.1 \text{ ft/sec}$$

$$h_v = 0.72 \text{ foot}$$

and h = 1.17 feet

Further refinement is unnecessary.

Thus, the average water surface upstream from the underpass is 1.2 feet higher than the tail water which satisfies the assumed design requirement of a maximum backwater of 15 inches. The length of the underpass is $2D_2$ or 20 feet, and the waves are reduced 80 percent to a maximum height of approximately 1 foot.

If it is desired to reduce the wave heights still further, a longer underpass is required. Using Table 7 and Figure 29B as in the above problem, an underpass 3.5 to $4.0D_2$ or 35 to 40 feet in length reduces the waves 90 to 93 percent, making the downstream waves approximately 0.5 foot high and creating a backwater, h, of 1.61 feet.

In using the above heads, allowance should be made for waves and surges which, in effect, are above the computed water surface. One-half the wave height or more, measured from crest to trough, should be allowed above the computed surface. Full wave height would provide a more conservative design for the usual short period waves encountered in flow channels.

The headwall of the underpass should be extended to this same height and a seawall overhang placed at the top to turn wave spray back into the basin. An alternate method would be to place a cover, say $2D_2$ long, upstream from the underpass headwall.

To insure obtaining the maximum wave reduction for a given length of underpass, a 4:1 sloping roof should be provided at the downstream end of the underpass, as indicated on Figure 28. This slope may be considered as part of the over-all length. The sloping roof will help reduce the maximum wave height and will also reduce the frequency with which it occurs, providing in all respects a better appearing water surface.

A close inspection of the submitted data will reveal that slightly better results were obtained in the tests than are claimed in the example. This was done to illustrate the degree of conservatism required, since it should be understood that the problem of wave reduction can be very complex if unusual conditions prevail.

The data and sample problem given here are for conditions within the limits described. From these data it should be possible to design a wave suppressor for general use with a good degree of accuracy. Care should be taken, however, that the data are not extended beyond the limits given. When any doubt exists, a model study should be made, particularly if the wave reduction must be accomplished because of a measuring device located immediately downstream from the suppressor. Additional model tests will be required to be certain that the limited amount of data, from which these conclusions were drawn, represent typical problems encountered in the design of field structures.