UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

PROGRESS REPORT II

RESEARCH STUDY ON STILLING BASINS ENERGY DISSIPATORS, AND ASSOCIATED APPURTENANCES

Hydraulic Laboratory Report No Hyd-399

ENGINEERING LABORATORIES

COMMISSIONER S OFFICE
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Commissioner's Office
Engineering Laboratories
Denver, Colorado
June 1, 1955

Laboratory Report No. Hyd-399*
Hydraulic Laboratory
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Subject: Progress Report II; Research Study on Stilling Basins, Energy Dissipators, and Associated Appurtenances

INTRODUCTION

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 overall 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.

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*Supersedes Laboratory Report No. Hyd-380; Progress Report, Research Study on Stilling Basins and Bucket Dissipators; dated June 11, 1954. J. N. Bradley is now Hydraulic Engineer, Bureau of Public Roads, Washington, D. C.

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 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 'atum, 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.

Although much of the original data are presented in tabular form, the report emphasizes design procedures rather than the hydraulic aspects of the data. Certain design procedures, recommended in the past, have been satisfactorily proven, 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 limits in broader terms, making it necessary to carefully read the accompanying text.

Proper use of the material in this report will eliminate the need for hydraulic model tests on many individual structures, particularly the smaller ones. Structures obtained by following the recommendations in this report 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.

SCOPE

The entire program, as planned, is listed below. The first six numbers correspond to the section numbers in this progress report. Items 7 through 13 will be completed as time and funds permit.

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.

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.

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.

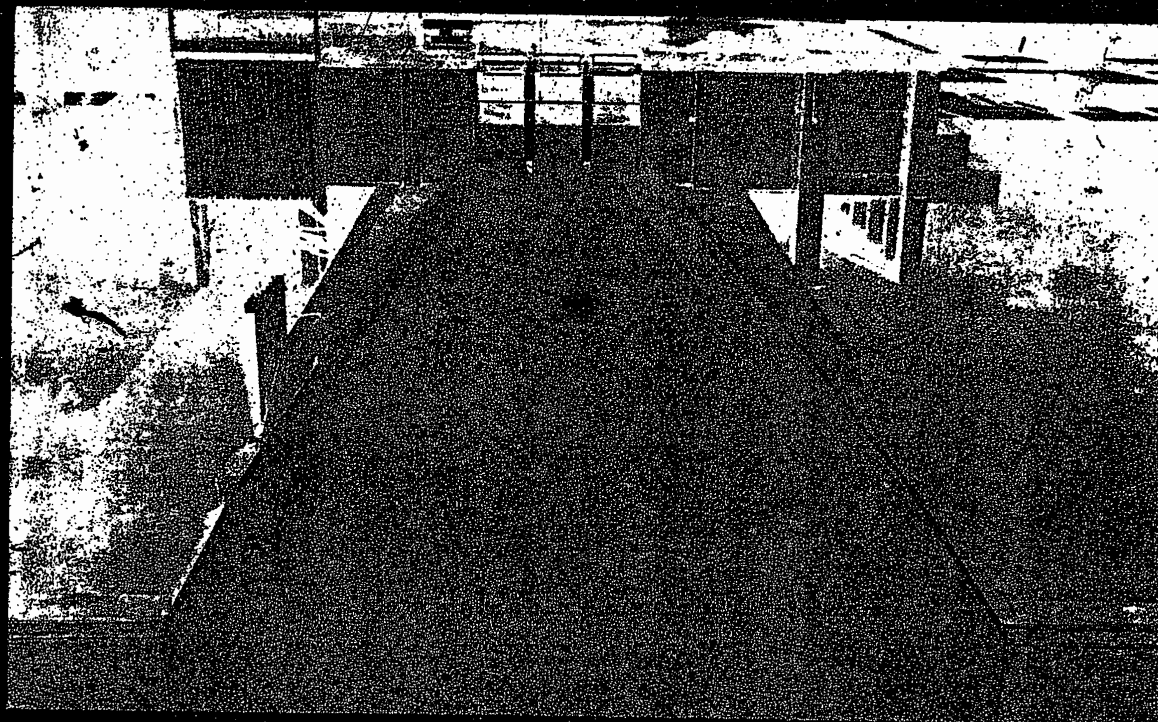
The experiments were started in an existing model of the Trenton Dam spillway, Figure 1A, having a small discharge and low velocity. This was not an ideal piece of equipment for general experiments as the training walls on the chute were diverging. The rapid expansion caused the distribution of flow entering the stilling basin to shift with each change in discharge; nonetheless, this piece of equipment served a purpose in that it aided in getting the research program underway.

Tests were then continued in a glass-sided laboratory flume 2 feet wide and 40 feet long in which an overflow section was installed, Flume B, Figure 1. The crest of the overflow section was 5.5 feet above the floor, while the downstream face was on a slope of 0.7:1. The capacity was about 10 cfs.

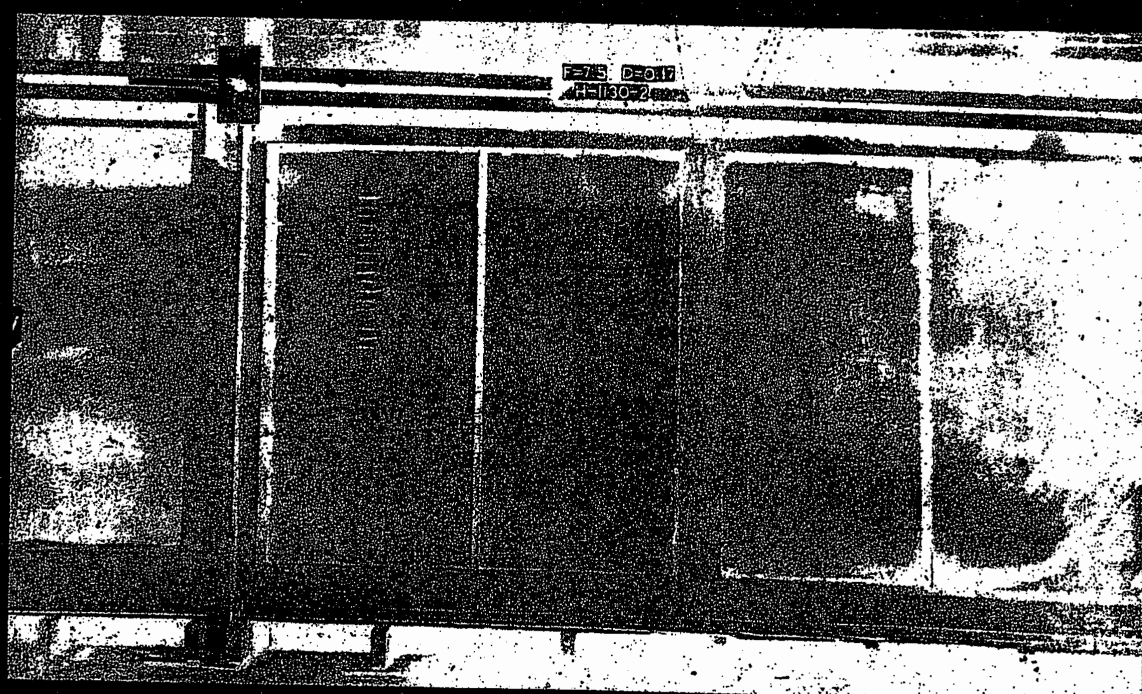
Later, the work was carried on at the base of a chute 18 inches wide having a slope of 2 horizontal to 1 vertical and a drop of approximately 10 feet, Flume C, Figure 2. The stilling basin had a glass wall on one side. The discharge capacity was 5 cfs.

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.

FIGURE 1

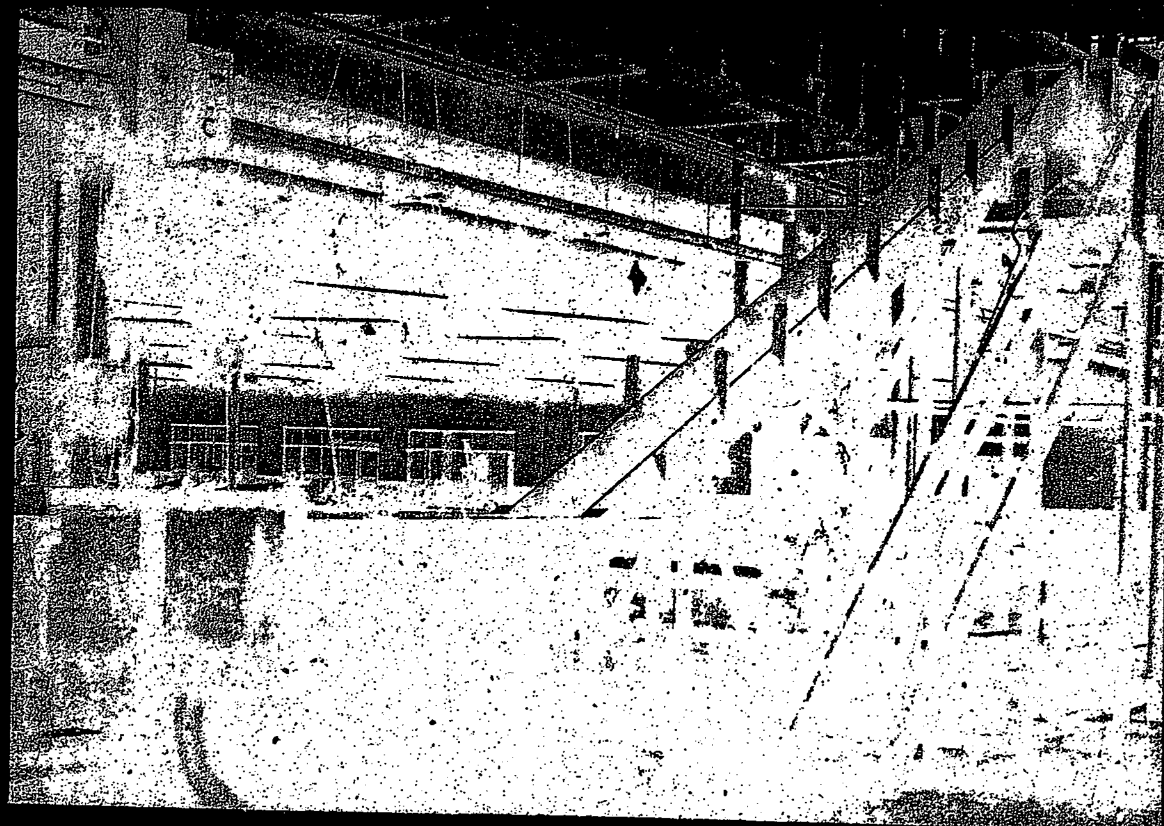


TEST FLUME A
Width of basin 5 feet, drop 3 feet, discharge 6 cfs



TEST FLUME B
Width 2 feet, drop 5.5 feet, discharge 10 cfs

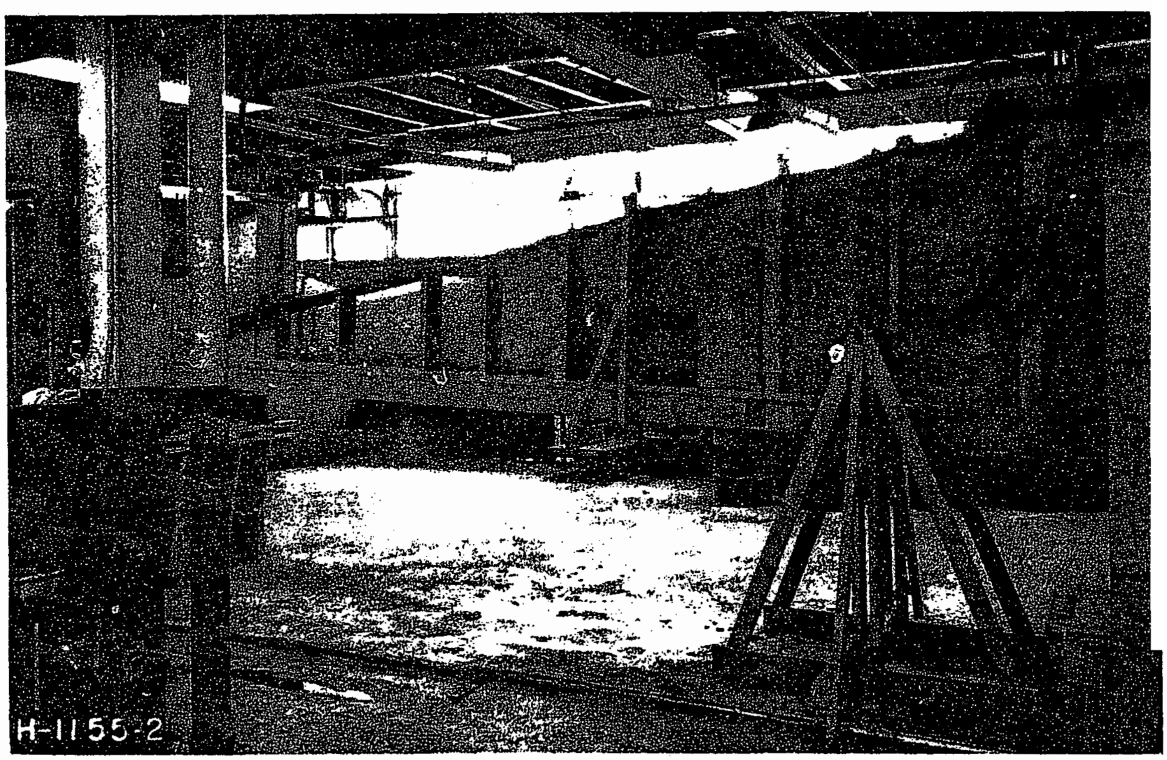
FIGURE 2



TEST FLUME C
Width 1.5 feet, drop 10 feet, discharge 5 cfs, slope 2:1

TEST FLUME D
Width 4 feet, drop 12 feet, discharge 28 cfs, slope 0.8:1

TEST FLUME (E)
Width 4 feet, Drop 0.5-1.5 feet, Discharge 10 cfs



TEST FLUME (F)

The downstream end of the above flume was also utilized for testing small overflow sections 0.5 to 1.5 feet in height. The maximum discharge used was 10 cfs. As stated above, this piece of equipment will be designated as Flume E, and is shown on Figure 3.

The sixth piece of equipment was a tilting flume which could be adjusted for slopes up to 12°, Flume F, Figure 3. This flume was 1 foot wide by 20 feet long; the head available was 2.5 feet, and the flow was controlled by a slide gate. The discharge capacity was about 3 cfs.

Each piece of equipment contained a head gage, a tail gage, a scale for measuring the length of the jump, a point gage for measuring the average depth of flow entering the jump, and a means of regulating the tail water depth. The discharge in all cases was measured through the laboratory venturi meters or portable venturi-orifice meters. The tail water depth was measured by a point gage operating in a stilling-well in most of the cases. The tail water depth was regulated by an adjustable weir at the end of each flume.

REMARKS

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.

SECTION 1

GENERAL INVESTIGATION OF THE HYDRAULIC JUMP ON HORIZONTAL APRON (BASIN I)

Introduction

A tremendous amount of experimental, as well as theoretical, work has been performed in connection with the hydraulic jump on a horizontal apron. To mention a few of the experimenters who contributed basic information, there are: Bakhmeteff and Matzke[1 9], Safranez[3], Woycicki[4], Chertonosov[10], Einwachter[11], Elms[12], Hinds[14], Forcheimer[21], Kennison[22], Kozeny[23], Rehbock[24], Schoklitsch[25], Woodward[26], and others. 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 overall 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.

Current Experimentation

To satisfactorily observe the hydraulic jump throughout its entire range required a testing program in all of the six flumes shown on Figures 1, 2, and 3. This involved about 125 tests, Table 1, at discharges from 1 to 28 cfs. The number of flumes used enhanced the value of the results in that it was possible to observe the degree of similitude obtained for the various scales. Greatest reliance was naturally placed on the results from the larger scales, or larger flumes, as it is well known that the jump action in small models occurs too rapidly for the eye to follow details. Incidentally, the length of jump obtained from the two smaller flumes, A and F, was consistently shorter than that observed for the larger flumes. This was the result of out-of-scale frictional resistance on the floor and side walls. As testing advanced and this deficiency became better understood, some allowance was made for this effect in the observations.

Table 1

NATURAL STILLING BASIN WITH HORIZONTAL FLOOR
(Stilling Basin I)

Test flume (1)	$S = \tan\alpha$ Slope of floor (2)	Q Total discharge cfs (3)	W Width of stilling basin ft (4)	q Discharge per ft of basin width (5)	TW $= D_2$ Tail water depth ft (6)	V_1 Velocity entering jump ft/sec (7)	D_1 Depth entering jump ft (8)	$\frac{D_2}{D_1}$ (9)	$F_1 = \frac{V_1}{\sqrt{gD_1}}$ (10)	L Length of jump ft (11)	$\frac{L}{D_2}$ (12)	$\frac{L}{D_1}$ (13)	$E_1 = d_1 + \frac{V_1^2}{2g}$ ft (14)	$E_2 = d_2 + \frac{V_2^2}{2g}$ ft (15)	$E_L = E_1 - E_2$ ft (16)	$\frac{E_L}{D_1}$ (17)	$\frac{E_L}{E_1}$ % (18)	Gate opening ft (19)
A	0	3.000	4.915	0.610	0.564	8.47	0.072	7.83	5.56	3.0	5.32	42	1.187	0.582	0.605	8.40	51.0	
		3.500		0.712	0.612	8.79	.081	7.56	5.44	3.3	5.39	41	1.282	0.633	0.649	8.01	50.6	
		4.000		0.814	0.651	8.95	.091	7.15	5.23	3.5	5.38	38	1.336	0.675	0.661	7.26	49.5	
		4.500		0.916	0.694	8.98	.102	6.80	4.96	3.8	5.48	37	1.354	0.721	0.633	6.21	46.8	
		5.000		1.017	0.730	9.08	.112	6.52	4.78	4.1	5.62	37	1.392	0.760	0.632	5.64	45.4	
B	0	4.300	2.000	2.150	1.480	17.48	.123	12.03	8.78	9.0	6.08	73	4.868	1.513	3.355	27.28	68.9	
		5.000		2.500	1.600	17.48	.143	11.19	8.15	10.1	6.31	71	4.888	1.638	3.250	22.73	66.5	
		3.000		1.500	1.236	17.24	.087	14.20	10.30	7.5	6.07	86	4.702	1.259	3.443	39.57	73.2	
		6.000		3.000	1.754	17.54	.171	10.26	7.47	10.6	6.04	62	4.948	1.799	3.149	18.42	63.6	
		7.000		3.500	1.908	17.59	.199	9.59	6.95	11.5	6.03	58	5.003	1.960	3.043	15.29	60.8	
		8.000		4.000	2.016	17.47	.229	8.80	6.43	12.3	6.10	54	4.968	2.077	2.891	12.62	58.2	
		7.000		3.500	1.900	17.41	.201	9.45	6.84	11.5	6.05	57	4.908	1.953	2.955	14.70	60.2	
		3.110		1.550	1.240	17.61	.088	14.09	10.46	7.9	6.37	90	4.903	1.264	3.639	41.35	74.2	
		4.100		2.050	1.440	17.52	.117	12.31	9.03	8.7	6.04	74	4.883	1.471	3.412	29.16	69.9	
		5.975		2.988	1.760	17.58	.170	10.35	7.51	10.2	5.80	60	4.969	1.805	3.164	18.61	63.7	
		7.010		3.505	1.900	17.52	.200	9.50	6.90	11.3	5.95	56	4.967	1.953	3.014	15.07	60.7	
		8.000		4.000	2.030	17.54	.228	8.90	6.48	12.6	6.21	55	5.006	2.090	2.916	12.79	58.3	
		4.150		2.075	1.450	17.44	.119	12.18	8.91	8.8	6.07	74	4.842	1.482	3.360	28.24	69.4	
		5.500		2.750	1.691	17.40	.158	10.70	7.72	10.7	6.33	68	4.860	1.732	3.128	19.80	64.4	
		6.000		3.000	1.764	17.44	.172	10.26	7.41	11.0	6.24	64	4.895	1.809	3.086	17.94	63.0	
		6.500		3.250	1.827	17.38	.187	9.77	7.08	11.3	6.18	60	4.878	1.876	3.002	16.05	61.5	
		2.000		1.000	1.000	17.24	.058	17.24	12.62	6.4	6.40	110	4.674	1.016	3.658	63.07	78.3	
		2.500		1.250	1.104	17.36	.072	15.33	11.40	6.7	6.07	93	4.752	1.124	3.628	50.39	76.3	
		3.000		1.500	1.235	17.44	.086	14.36	10.48	7.4	5.99	86	4.809	1.258	3.551	41.29	73.8	
		3.500		1.750	1.325	17.50	.100	13.25	9.75	7.8	5.89	78	4.856	1.352	3.504	35.04	72.2	
		4.000		2.000	1.433	17.39	.115	12.46	9.04	8.5	5.93	74	4.811	1.463	3.348	29.11	69.6	
		4.500		2.250	1.517	17.44	.129	11.76	8.56	9.4	6.20	73	4.852	1.551	3.301	25.59	68.0	
		5.000		2.500	1.599	17.24	.145	11.03	7.98	10.0	6.25	69	4.761	1.637	3.124	21.54	65.6	
		5.500		2.750	1.691	17.40	.158	10.70	7.72	10.7	6.33	68	4.860	1.732	3.128	19.80	64.4	
C	0	1.000	1.500	0.667	0.910	20.21	.033	27.58	19.55	5.0	5.49	152	6.375	0.918	5.457	165.36	85.6	
		1.500		1.000	1.125	20.43	.048	23.44	16.76	6.4	5.69	133	6.785	1.137	5.648	117.67	83.2	
		2.000		1.333	1.300	21.16	.063	20.63	14.86	7.7	5.92	122	7.016	1.316	5.700	90.48	81.2	
		2.500		1.667	1.465	21.39	.078	18.78	13.48	8.9	6.08	114	7.169	1.485	5.684	72.87	79.3	
		3.000		2.000	1.615	21.74	.092	17.55	12.63	9.7	6.01	105	7.431	1.639	5.792	62.96	77.9	
		3.380		2.253	1.730	22.09	.102	16.96	12.19	10.8	6.24	106	7.679	1.756	5.923	58.07	77.1	
		4.000		2.667	1.890	22.79	.117	16.15	11.74	11.1	5.87	95	8.152	1.921	6.261	53.51	76.5	
		4.440		2.960	2.010	23.13	.128	15.70	11.39	12.3	6.11	96	8.435	2.044	6.391	49.93	75.8	
		1.250		0.833	0.914	17.35	.048	19.04	13.96	5.4	5.91	112	4.723	0.927	3.796	79.08	80.4	
		1.750		1.167	1.135	18.82	.062	18.30	13.32	6.5	5.73	105	5.563	1.151	4.412	71.16	79.3	
		2.250		1.500	1.320	19.48	.077	17.14	12.37	7.8	5.91	[illegible]	5.969	1.340	4.629	60.12	77.6	
		2.750		1.833	1.468	20.37	.090	16.31	11.97	9.1	6.20	101	6.533	1.492	5.041	56.01	77.1	
		3.250		2.167	1.616	20.84	.104	15.54	11.39	10.0	6.19	96	6.849	1.644	5.205	50.05	76.0	
		3.750		2.500	1.736	21.19	.118	14.71	10.87	11.0	6.34	93	7.091	1.768	5.323	45.11	75.1	
		4.250		2.833	1.870	21.14	.134	13.96	10.18	11.6	6.20	87	7.074	1.905	5.169	38.57	73.1	
D	0	4.000	3.970	1.008	1.110	20.16	.050	22.20	15.89	[illegible]	5.86	130	6.361	1.123	5.238	104.76	82.3	
		5.000		1.259	1.220	20.31	.062	19.68	14.37	7.5	[illegible]	121	6.467	1.236	5.231	84.37	80.9	
		6.000		1.511	1.376	20.42	.074	18.19	13.23	8.4	6.24	114	6.549	1.365	5.184	70.05	79.2	
		7.000		1.763	1.460	20.30	.086	16.98	12.32	9.0	6.16	105	6.612	1.483	5.129	59.64	77.6	
		8.000		[illegible]	1.570	20.56	.098	16.02	11.55	9.7	6.18	99	6.662	1.595	5.067	51.70	76.1	
		9.000		2.267	1.670	20.80	.109	15.32	11.11	10.0	5.99	92	6.827	1.699	5.128	47.05	75.1	
		8.080		2.035	1.600	20.56	.099	16.16	11.52	9.5	5.94	96	6.663	1.625	5.038	50.89	75.6	
		11.730		2.955	1.962	21.41	.138	14.22	10.16	12.4	6.32	90	7.256	1.997	5.259	38.11	[illegible]	
		10.000		2.519	1.732	20.99	.120	14.60	10.68	10.4	5.94	87	6.961	1.784	5.177	43.14	[illegible]	
		3.000		0.756	0.954	19.89	.038	25.11	12.98	5.4	5.66	142	6.181	0.964	5.217	137.29	84.4	
		5.000		1.259	1.250	20.31	.062	20.16	14.37	7.4	5.92	119	6.467	1.266	5.201	83.89	80.4	
		7.000		1.763	1.452	20.50	.086	16.88	12.12	8.7	5.99	101	6.612	1.475	5.137	59.73	77.7	
		9.000		2.267	1.693	20.80	.109	15.53	11.11	10.5	6.20	96	6.827	1.721	5.106	46.84	74.8	
		11.720		2.952	1.922	21.39	.138	13.93	10.15	11.4	5.93	83	7.243	1.959	5.284	38.29	73.0	
		10.000		2.519	1.780	20.99	.120	14.83	10.68	11.2	6.29	93	6.961	1.811	5.150	42.92	74.0	
		12.000		3.023	1.953	21.29	.142	13.75	9.96	12.3	6.30	87	7.180	1.990	5.190	36.55	72.3	
		14.000		3.526	2.163	21.63	.163	13.27	9.44	13.0	6.01	80	7.428	2.204	5.224	32.05	70.3	
		16.000		4.030	2.330	22.02	.183	12.73	9.07	13.8	5.92	75	7.712	2.376	5.336	29.16	69.2	
		18.000		4.534	2.495	22.56	.201	12.41	8.87	15.4	6.17	77	[illegible]	2.546	5.558	27.65	68.6	

Table 1--Continued

NATURAL STILLING BASIN WITH HORIZONTAL FLOOR
(Stilling Basin I)

Test flume	S = tan α: Slope of floor	Q Total discharge cfs	W Width of stilling basin ft	q Discharge per ft of basin width	D_2 = TW Tail water depth ft	V_1 Velocity entering jump ft/sec	D_1 Depth entering jump ft	$\frac{D_2}{D_1}$	$F_1 = \frac{V_1}{\sqrt{gD_1}}$	L Length of jump ft	$\frac{L}{D_2}$	$\frac{L}{D_1}$	$E_1 = d_1 + \frac{V_1^2}{2g}$ ft	$E_2 = d_2 + \frac{V_2^2}{2g}$ ft	$E_L = E_1 - E_2$ ft	$\frac{E_L}{D_1}$	$\frac{E_L}{E_1}$	Gate opening ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
D	0	4.980	3.970	1.254	1.220	20.23	.062	19.68	14.32	7.0	5.74	113	6.417	1.236	5.181	83.56	80.7	
		10.000		2.519	1.792	20.99	.120	14.93	10.68	11.0	6.14	92	6.961	1.823	5.138	42.82	73.8	
		11.000		2.771	1.867	21.15	.131	14.25	10.30	11.3	6.05	86	7.077	1.901	5.176	39.51	73.2	
		13.000		3.275	2.009	21.55	.152	13.22	9.74	12.4	6.17	82	7.363	2.050	5.313	34.95	72.2	
		15.000		3.778	2.180	21.84	.173	12.60	9.25	13.3	6.10	77	7.580	2.226	5.354	30.95	70.6	
		6.500		1.637	1.412	20.46	.080	17.65	12.75	8.9	6.30	111	6.580	1.433	5.147	64.34	78.2	
		4.980		1.254	1.220	20.23	.062	19.68	14.32	7.0	5.74	113	6.417	1.236	5.181	83.56	80.7	
		17.000		4.282	2.410	22.30	.192	12.56	8.77	14.6	6.05	76	7.914	2.461	5.453	28.40	68.9	
		19.000		4.786	2.560	22.79	.210	12.19	8.77	15.3	5.98	73	8.275	2.614	5.661	26.96	68.4	
		21.000		5.290	2.656	23.20	.228	11.65	8.56	16.0	6.02	70	8.586	2.717	5.869	25.74	68.4	
		26.160		6.589	3.060	24.22	.272	11.25	8.19	19.4	6.34	71	9.381	3.132	6.249	22.97	66.6	
		22.980		5.788	2.842	24.12	.240	11.84	8.68	18.7	6.58	78	9.274	2.907	6.367	26.53	68.7	
		23.930		6.028	2.845	23.73	.254	11.20	8.30	18.3	6.43	72	8.998	2.915	6.083	23.95	67.6	
		28.370		7.147	3.202	24.56	.291	11.00	8.02	21.0	6.56	72	9.657	3.279	6.378	21.92	66.0	
F	0	0.960	1.000	0.960	0.792	12.15	.079	10.03	7.62	4.0	5.05	51	2.371	0.815	1.556	19.70	65.6	0.125
		0.815		0.815	0.653	9.59	.085	7.68	5.80	3.0	4.59	35	1.513	0.677	0.836	9.84	55.3	
		0.680		0.680	0.540	8.61	.079	6.84	5.40	2.4	4.44	30	1.230	0.565	0.665	8.42	54.1	
		1.580		1.580	0.992	11.70	.135	7.35	5.61	6.2	6.25	46	2.261	1.031	1.230	9.11	54.4	.202
		1.200		1.200	0.740	8.89	.135	5.48	4.26	4.3	5.81	32	1.362	0.781	0.581	4.30	42.7	
		1.400		1.400	0.880	10.37	.135	6.52	4.97	5.4	6.14	40	1.805	0.919	0.886	6.56	49.1	
		2.230		2.230	1.220	12.89	.173	7.05	5.46	7.3	5.98	42	2.753	1.272	1.481	8.56	53.8	.281
		1.730		1.730	0.927	10.00	.173	5.36	4.24	5.2	5.61	30	1.726	0.981	0.745	4.31	43.2	
		1.250		1.250	0.644	7.23	.173	3.72	3.06	3.4	5.28	20	0.985	0.702	0.283	1.64	28.7	
		1.150		1.150	0.581	6.65	.173	3.36	2.82	3.1	5.34	18	0.860	0.642	0.218	1.26	25.3	
		1.400		1.400	0.638	6.69	.210	3.04	2.57	3.3	5.17	16	0.901	0.712	0.189	0.90	21.0	.333
		1.850		1.850	0.882	8.81	.210	4.20	3.39	5.0	5.67	24	1.415	0.950	0.465	2.21	32.9	
		2.200		2.200	1.075	10.48	.210	5.12	4.03	6.2	5.77	30	1.915	1.140	0.775	3.69	40.5	
		2.740		2.740	1.345	13.05	.210	6.40	5.02	8.2	6.10	39	2.854	1.410	1.444	6.88	50.6	
		1.830		1.830	0.753	7.29	.251	3.00	2.56	3.5	4.65	14	1.076	0.845	0.231	0.92	21.5	.396
		2.350		2.350	1.023	9.36	.251	4.08	3.29	5.6	5.47	22	1.611	1.105	0.506	2.02	31.4	
		2.780		2.780	1.235	11.08	.251	4.92	3.90	7.2	5.83	29	2.157	1.314	0.843	3.36	39.1	
		3.165		3.165	1.427	12.61	.251	5.69	3.44	8.4	5.89	33	2.720	1.504	1.216	4.84	44.7	
		1.900		1.900	0.704	6.67	.285	2.47	2.20	3.2	4.55	11	0.976	0.817	0.159	0.56	16.3	.458
		2.540		2.540	1.016	8.91	.285	3.56	2.94	5.3	5.22	19	1.518	1.113	0.405	1.42	26.7	
		3.000		3.000	1.219	10.53	.285	4.28	3.48	7.0	5.74	25	2.007	1.313	0.694	2.44	34.6	
		3.460		3.460	1.435	12.14	.285	5.04	4.01	8.3	5.78	29	2.574	1.525	1.049	3.68	40.8	
E	0	5.000	3.970	1.259	0.840	10.49	0.120	7.00	5.34	5.0	5.95	42	1.831	0.875	0.956	7.97	52.2	Dam 1.5
		6.000		1.511	0.940	10.57	.143	6.57	4.92	5.6	5.96	39	1.880	0.980	.900	6.29	47.9	high
		7.000		1.763	0.990	10.75	.164	6.04	4.67	5.9	5.96	36	1.960	1.039	.921	5.62	47.0	
		8.000		2.014	1.080	10.89	.185	5.84	4.46	6.3	5.83	34	2.029	1.134	.895	4.84	44.1	
		9.000		2.266	1.160	11.05	.205	5.66	4.30	6.6	5.69	28	2.104	1.219	.885	4.32	42.1	
		11.000		2.770	1.260	11.17	.248	5.08	3.95	7.1	5.63	32	2.188	1.335	.853	3.44	39.0	
		4.000		1.008	0.770	10.28	.098	7.86	5.79	4.7	6.10	48	1.742	0.796	.946	9.65	54.3	
		10.000		2.518	1.220	11.09	.227	5.37	4.10	6.9	5.66	30	2.139	1.286	.853	3.76	39.9	
	0	10.002	3.970	2.518	1.080	8.99	.280	3.86	3.00	6.0	5.56	21	1.536	1.164	.372	1.33	24.2	Dam 10"
		9.000		2.266	1.000	8.78	.258	3.88	3.05	5.5	5.50	21	1.457	1.080	.377	1.46	25.9	high
		8.000		2.014	0.960	8.76	.230	4.17	3.22	5.0	5.21	22	1.413	1.029	.384	1.67	27.2	
		7.000		1.763	0.900	8.24	.214	4.21	3.13	4.7	5.22	22	1.269	0.961	.308	1.44	24.3	
		6.000		1.511	0.820	8.39	.180	4.56	3.48	4.3	5.24	24	1.274	0.873	.401	2.23	31.5	
		5.000		1.259	0.760	7.77	.162	4.69	3.40	4.1	5.39	25	1.102	0.803	.299	1.85	27.2	
		4.000		1.007	0.660	7.75	.130	5.08	3.79	3.7	5.61	28	1.064	0.697	.367	2.82	34.5	
		3.000		0.755	0.570	7.95	.100	5.70	4.21	3.3	5.79	33	1.082	0.597	.485	4.85	44.8	
	0	5.084	3.970	1.281	0.620	5.80	.221	2.81	2.17	2.6	4.19	12	0.744	0.686	.058	0.26	7.80	
		3.675		0.926	0.510	5.12	.168	3.04	2.20	2.5	4.90	15	0.576	0.561	.015	0.09	2.60	Dam 6"
		2.440		0.615	0.410	5.44	.113	3.63	2.85	2.2	5.36	19	0.573	0.445	.128	1.13	22.30	high
		7.680		1.934	0.770	5.69	.340	2.26	1.72	3.0	3.90	9	0.874	0.866	.008	0.02	0.92	
		6.000		1.511	0.690	5.68	.266	2.59	1.93	2.8	4.06	10	0.768	0.765	.003	0.01	0.39	

Experimental Results

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 by averaging a generous number of point gage measurements taken immediately upstream from Section 1, Figure 4. The results of the measurements and succeeding computations are tabulated in Table 1. The measured quantities are tabulated as follows: total discharge (Column 3), tail water depth, which should be the conjugate depth in this case (Column 6), length of jump (Column 11), and depth of flow entering jump (Column 8).

Column 1 indicates the test flumes in which the experiments were performed, and Column 4 shows the width of each flume. All computations are based on discharge per foot width of flume, or q, and unit discharges are shown in Column 5.

The velocity entering the jump V_1, Column 7, was computed by dividing q (Column 5) by D_1 (Column 8).

The Froude Number

The Froude number, Column 10, Table 1, is simply:

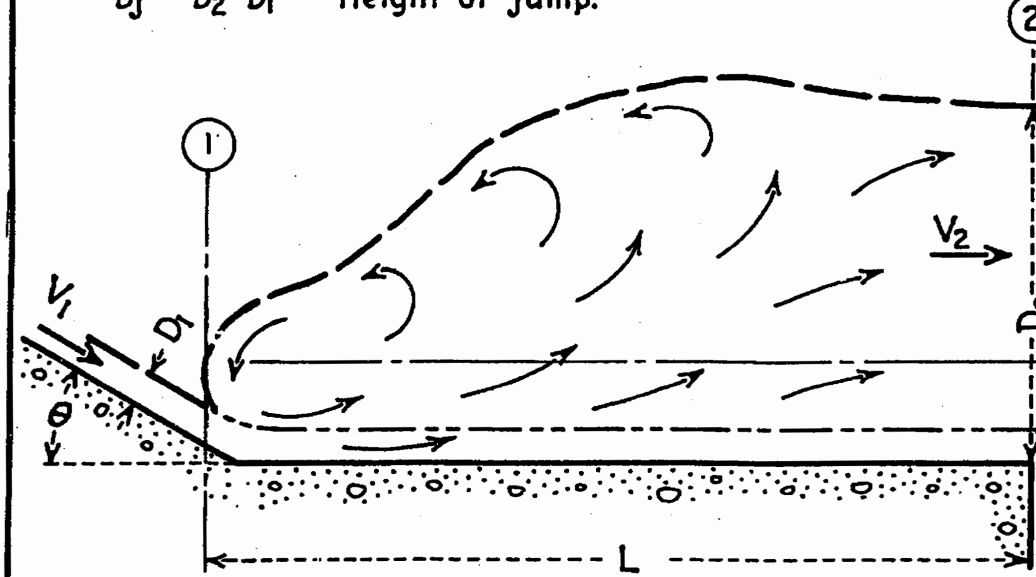
$$F_1 = \frac{V_1}{\sqrt{gD_1}} \qquad (1)$$

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. Bahkmeteff and Matzke[1] demonstrated this application in 1936 when they related stilling basin characteristics to the square of the Froude number, $\frac{V^2}{gD_1}$. They termed this expression the kinetic flow factor.

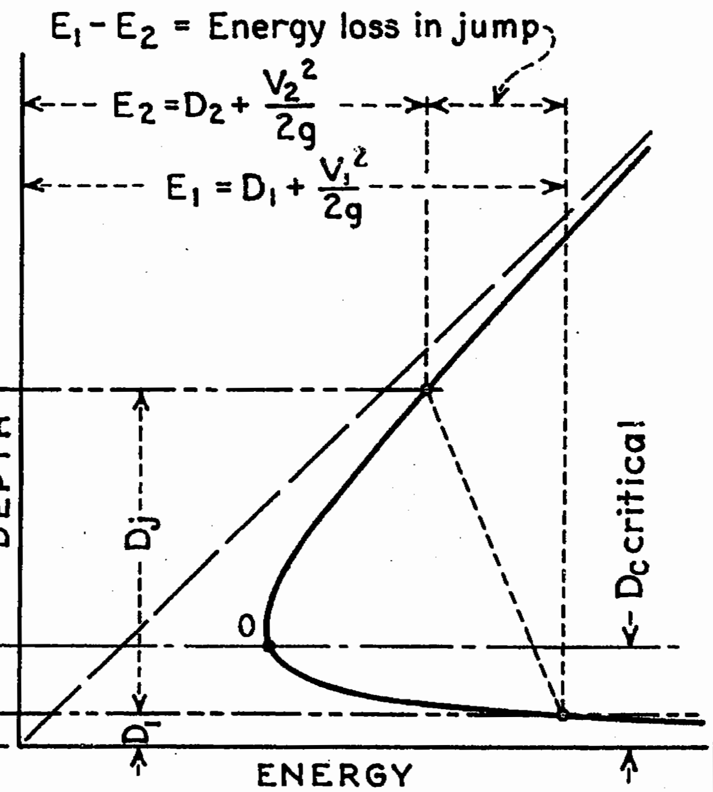
The Froude number will be used throughout this presentation. As the acceleration of gravity is a constant, the term g could be omitted. Its inclusion makes the expression dimensionless, however, and the form shown as (1) is preferred.

FIGURE 4

Q = Total discharge in c.f.s.
W = Width of flume in feet.
q = Discharge in c.f.s. per foot of width.
E_1 = Energy entering jump.
E_2 = Energy leaving jump.
F_1 = Froude number $- \frac{V_1}{\sqrt{g d_1}}$
D_j = $D_2 - D_1$ - Height of jump.



HYDRAULIC JUMP - ON HORIZONTAL FLOOR



RELATION OF SPECIFIC ENERGY TO DEPTH OF FLOW

HYDRAULIC JUMP STUDIES
STILLING BASIN I
DEFINITION OF SYMBOLS

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Applicability of Hydraulic Jump Formula

The theory of the hydraulic jump in horizontal channels has been treated thoroughly by others (see Bibliography), 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:[15]

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

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 $\frac{V_1^2}{gD_1}$,

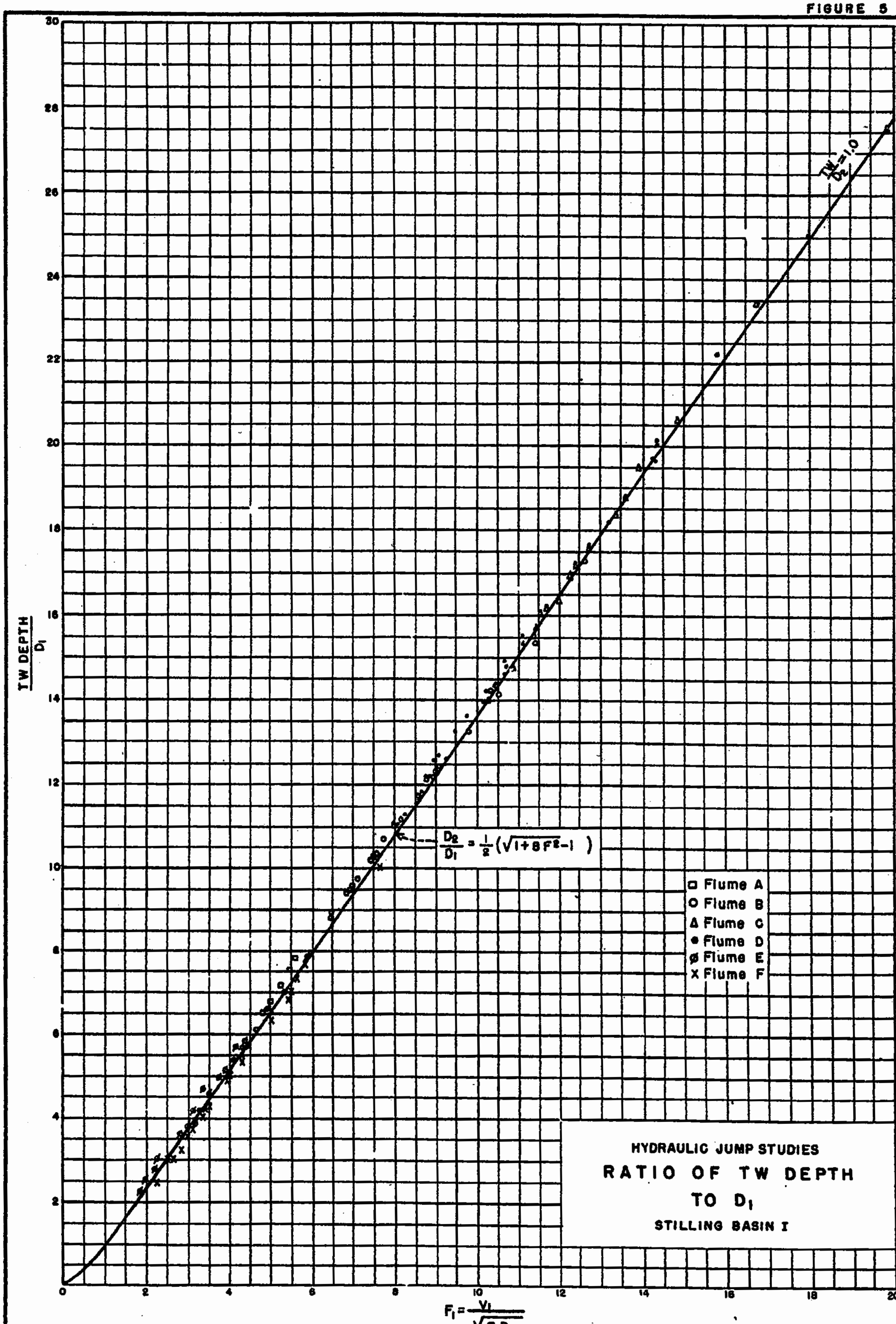
$$\frac{D_2}{D_1} = -1/2 + \sqrt{1/4 + 2F_1^2}$$

or

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

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, which are experimental, are from Columns 9 and 10, Table 1. The agreement is quite good for the entire range. There is an unsuspected characteristic, however, which should be mentioned here but will be enlarged on later.

Although the tail water depth, recorded in Column 6 of Table 1, 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.



HYDRAULIC JUMP STUDIES

RATIO OF TW DEPTH TO D_1

STILLING BASIN I

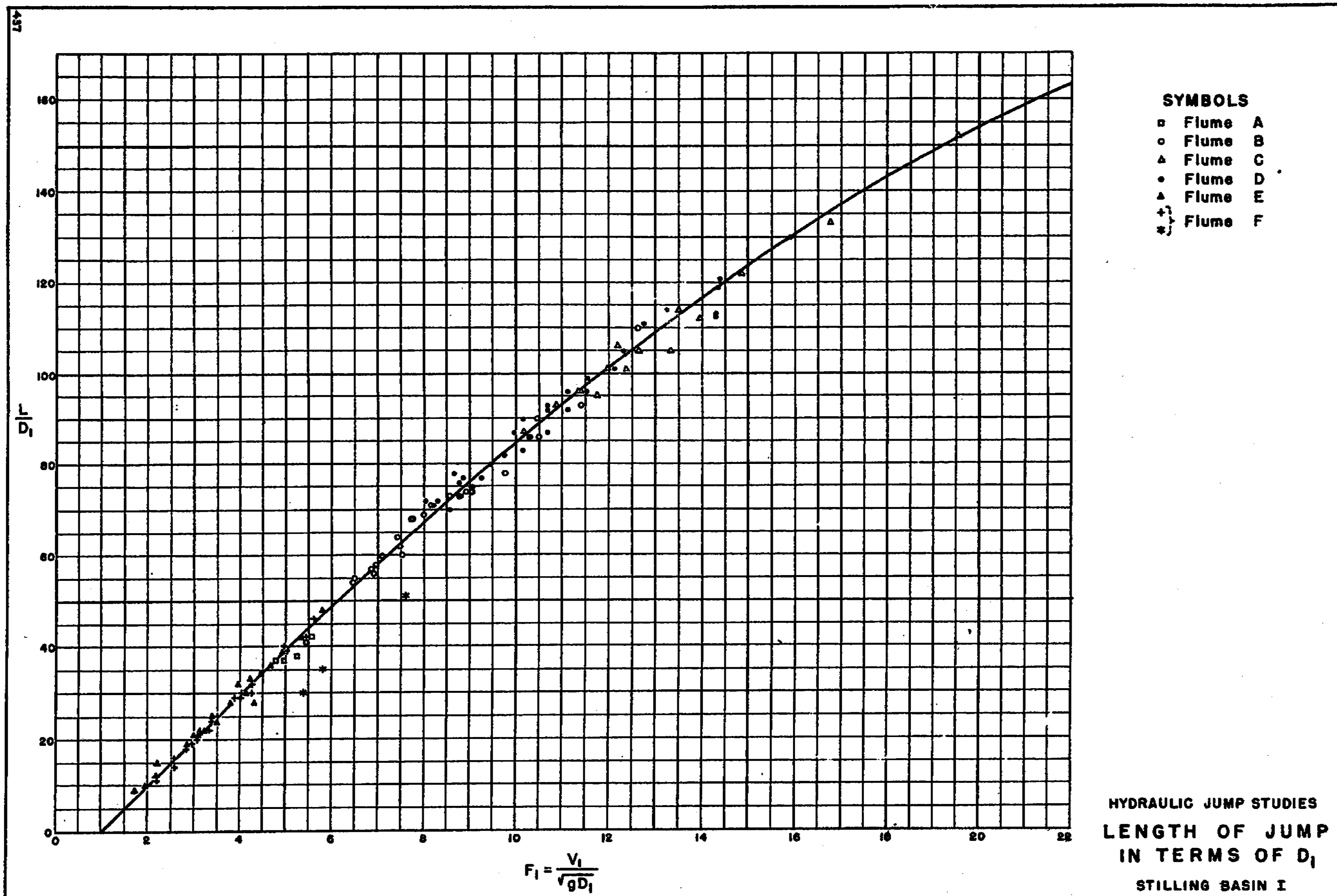
Length of Jump

The length of the jump, Column 11, Table 1, was the most difficult measurement to determine. 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. It was at first difficult to repeat length observations within 5 to 10 percent by either criterion, but with practice satisfactory measurements became possible.

A system devised to measure velocities on the bottom, to aid in determining the length of jump, proved inadequate and too laborious to allow completion of the program planned. Visual observations, therefore, proved to be the most satisfactory as well as the most rapid method for determining the length measurement. It was the intention to judge the length of the jump from a practical standpoint; other words, the end of the jump, as chosen, would represent the end of the concrete floor and side walls of a conventional stilling basin.

The length of jump has been plotted in two ways. The first is perhaps the better method while the second is the more common. The first method is shown on Figure 6 where the ratio length of jump to D_1 (Column 13, Table 1) is plotted with respect to the Froude number, (Column 10) for results from the six test flumes. The resulting curve is fairly flat, which is the principal advantage gained by the use of these coordinates. The second method of plotting, where the ratio of length of jump to the conjugate tail water depth D_2 (Column 12) is plotted with respect to the Froude number, is presented on Figure 7. This latter method of plotting will be 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 best known and most widely accepted curve for length of jump is that of Bakhmeteff and Matzke[1] which was determined from experiments made at Columbia University. The greater portion of this curve, labeled 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.



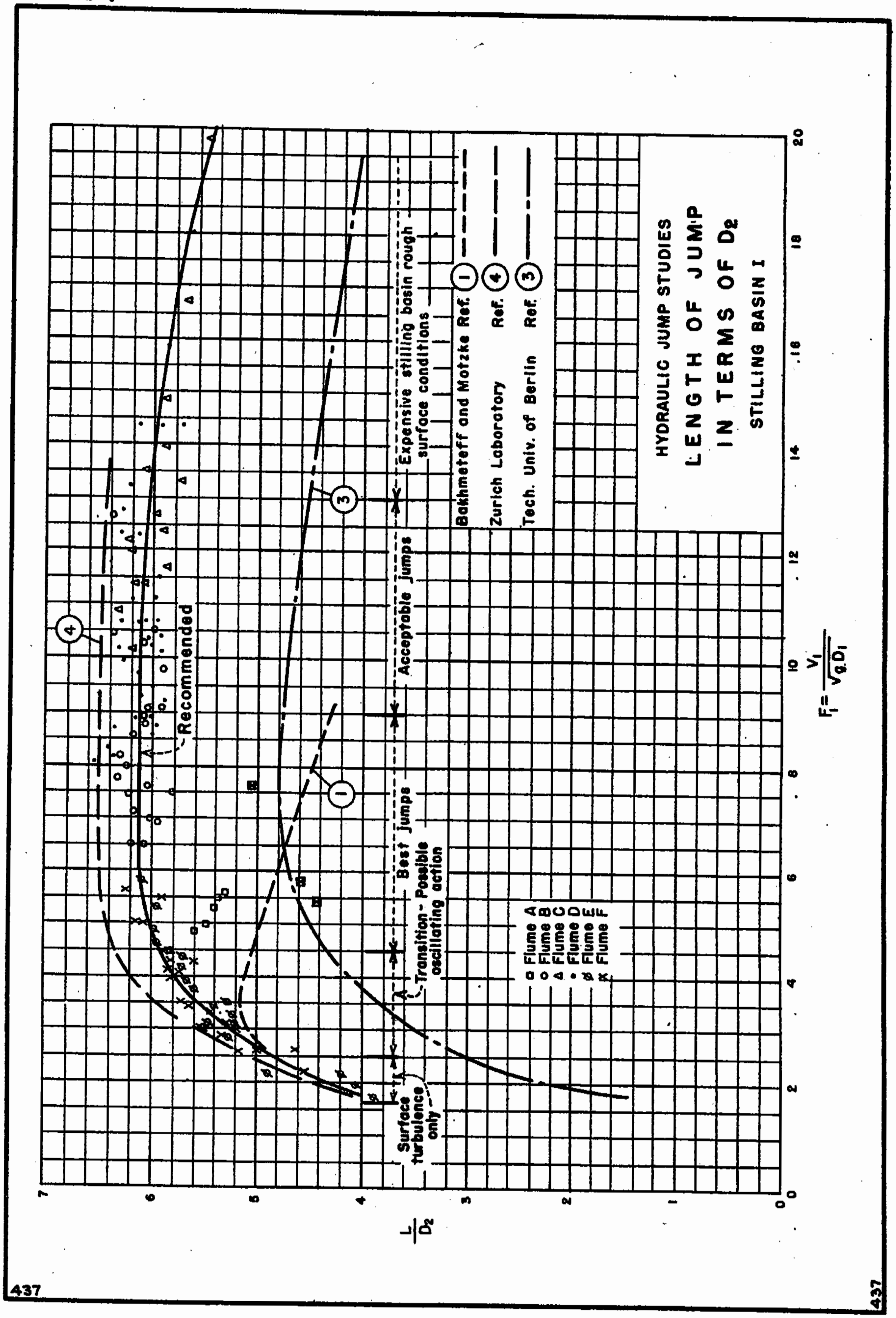
HYDRAULIC JUMP STUDIES

LENGTH OF JUMP IN TERMS OF D_1

STILLING BASIN I

FIGURE 6

FIGURE 7



HYDRAULIC JUMP STUDIES
LENGTH OF JUMP
IN TERMS OF D_2
STILLING BASIN I
Bakhmeteff and Motzke Ref. 1
Zurich Laboratory Ref. 4
Tech. Univ. of Berlin Ref. 3
□ Flume A
○ Flume B
Δ Flume C
• Flume D
ø Flume E
x Flume F
Surface turbulence only
Transition - Possible oscillating action
Best jumps
Acceptable jumps
Expensive stilling basin rough surface conditions
Recommended
$\frac{L}{D_2}$
$F_1 = \frac{V_1}{\sqrt{g D_1}}$
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
14
16
18
20
437
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The experiments of Bakhmeteff and Matzke were performed in a flume 6 inches wide, with limited head. The depth of flow entering the jump was adjusted by a vertical slide gate. 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. To increase the Froude number, it was necessary for Bakhmeteff and Matzke to decrease the gate opening. 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 as the gate opening decreased, 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. Thus, the jump formed in a shorter length than it should. In laboratory language, this is known as "scale effect," and is construed to mean that prototype action is not faithfully reproduced. It is quite certain that this was the case for the major portion of the Bahkmeteff-Matzke curve. In fact, they were somewhat dubious concerning the small scale experiments.

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. Figures 6 and 7 show three points obtained with a value of D_1 of approximately 0.085. The three points are given the symbol x 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. The curve labeled 3 was obtained at the Technical University of Berlin on a flume 1/2 meter wide by 10 meters long. The curve labeled 4 was determined from experiments performed at the Federal Institute of Technology, Zurich, Switzerland, on a flume 0.6 of a meter wide and 7 meters long. The curve numbers are the same as the reference numbers in the Bibliography which refer to the work.

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.

Energy Absorption in Jump

With the experimental information available, it is only a matter of computation to determine the energy absorbed in the jump. Columns 14 through 18, Table 1, list the computations, and the symbols may be defined by consulting the specific energy diagram on Figure 4. Column 14 lists the total energy, E_1, entering the jump at Section 1 for each test. This is simply the depth of flow, D_1, plus the velocity head computed at the point of measurement. The energy leaving the jump, which is the depth of flow plus the velocity head at Section 2, is tabulated in Column 15. The differences in the values of Columns 14 and 15 constitute the loss of energy, in feet of water, attributed to the conversion, Column 16. Column 18 lists the percentage of energy lost in the jump E_L, to total energy entering jump, E_1. This percentage is plotted with respect to 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 $\frac{E_L}{D_1}$ (Column 17, Table 1) plotted against the Froude number As there are those who prefer this method of plotting, the latter curve has been included.

Forms of the Hydraulic Jump

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 cataloged 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 O 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

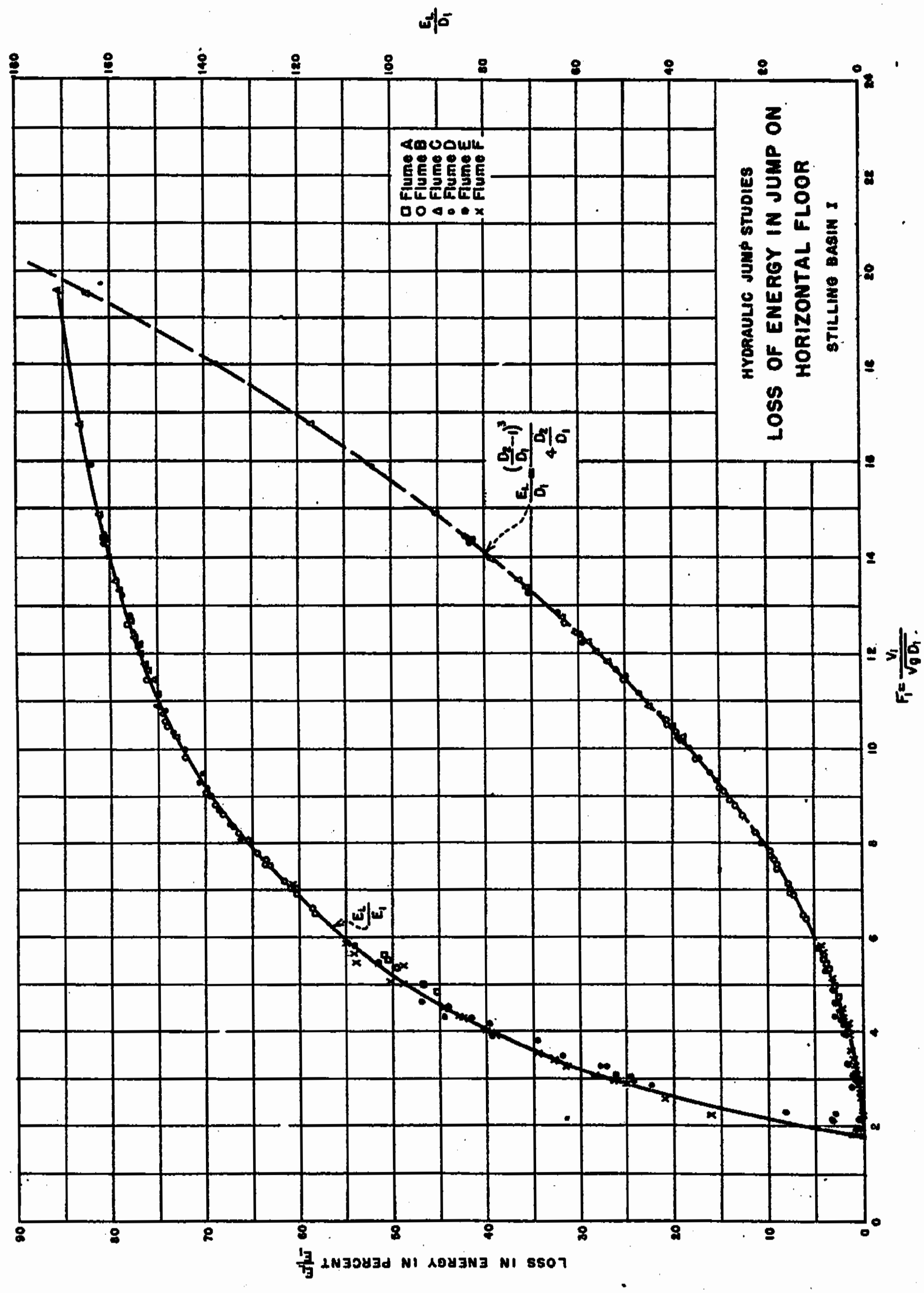
HYDRAULIC JUMP STUDIES
LOSS OF ENERGY IN JUMP ON HORIZONTAL FLOOR
STILLING BASIN I
□ Flume A
○ Flume B
△ Flume C
• Flume D
• Flume E
x Flume F
LOSS IN ENERGY IN PERCENT $\frac{E_L}{E_1}$
$F_1 = \frac{V_1}{\sqrt{g D_1}}$
$\frac{E_L}{D_1}$
$\frac{E_L}{D_1} = \frac{\left(\frac{D_2}{D_1}-1\right)^3}{4\frac{D_2}{D_1}}$
$\frac{E_L}{E_1}$

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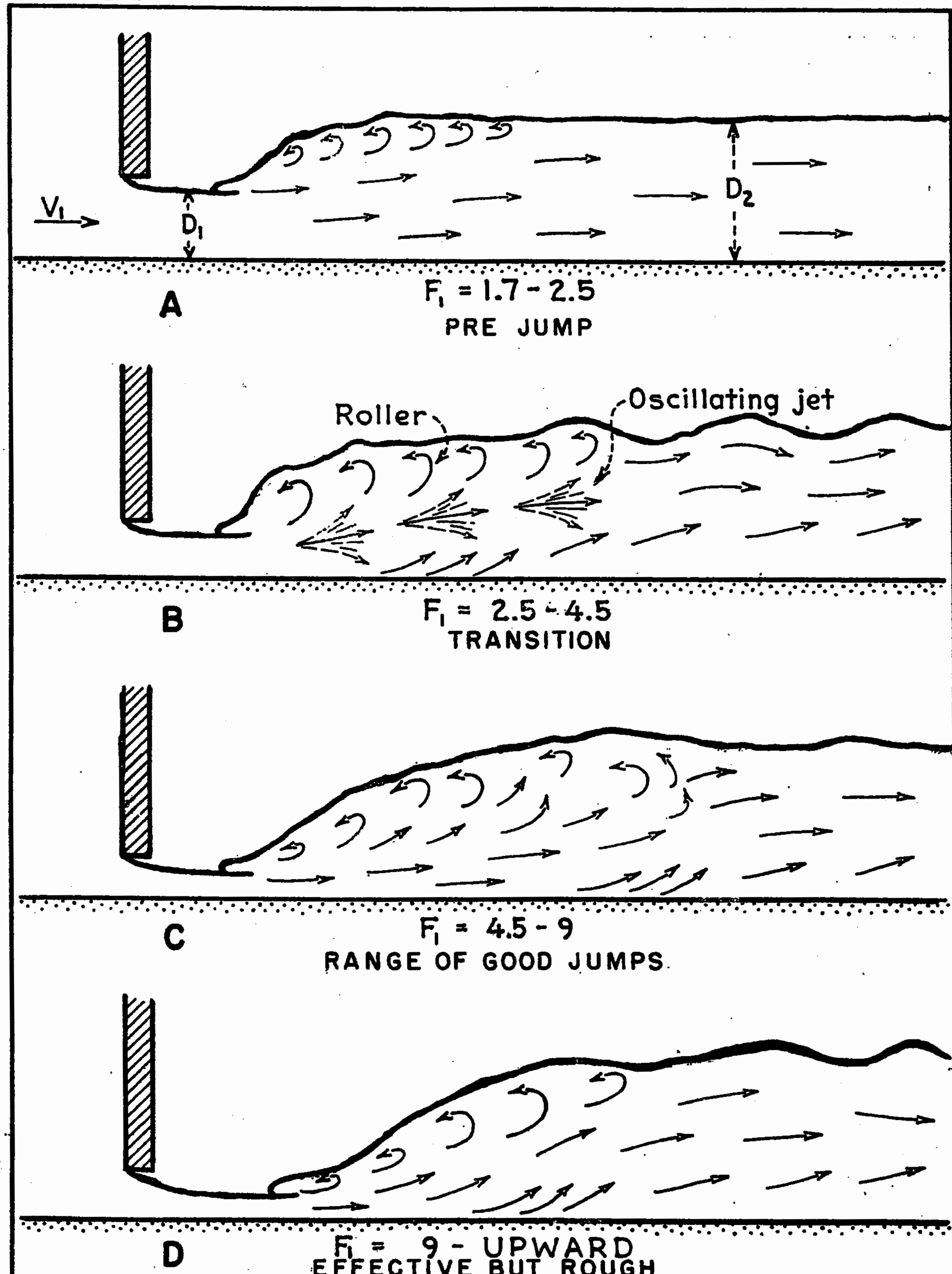
FIGURE 9



HYDRAULIC JUMP STUDIES

STILLING BASIN I

JUMP FORMS

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, Section 4, 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, and 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 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 in Section 4. 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 in the following sections.

6. When the Froude number is greater than 10, a stilling basin may no longer be the most economical dissipation device. The difference in conjugate depths is great, and, generally speaking, a very deep basin with high training walls is required. The cost of the stilling basin may not be commensurate with the results obtained. A bucket type of dissipator may give comparable results at less cost.

Water-surface Profiles and Pressures

Water-surface profiles for the jump on a horizontal floor were not measured as these have already been determined by Bakhmeteff and Matzke,[1] Newman and LaBoon,[19] and Moore.[27 18] It has been shown by several experimenters that the vertical pressures on the floor of the stilling basin are virtually the same as the water-surface profile would indicate. Although there will be more air entrainment and bulking in the prototype, making the freeboard of training walls less than indicated in the model, pressures obtained from models are sufficiently accurate for design purposes.

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.

Application of Results (Example 1)

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.

SECTION 2

STILLING BASIN FOR HIGH DAM AND EARTH DAM SPILLWAYS AND LARGE CANAL STRUCTURES (BASIN II)

INTRODUCTION

Stilling basins are seldom designed to confine the entire length of the hydraulic jump on the paved apron as was assumed in the foregoing section; first, for economic reasons and secondly, because there are means for modifying the jump characteristics to obtain comparable or better performance in shorter lengths. It is possible to reduce the jump length by the installation of accessories such as blocks, baffles, and sills in the stilling basin. In addition to shortening the jump, the accessories exert a stabilizing effect and in some cases increase the factor of safety.

Section 2 concerns stilling basins which have been in common use on high dam and earth dam spillways, and large canal structures, and will be denoted as Basin II (Figure 10). The basin contains chute blocks at the upstream end and a dentated sill near the downstream end. No baffle piers are used in Basin II because of the relatively high velocities entering the jump. The principal aim was to (1) generalize the design, and (2) determine the range of operating conditions for which this basin is best suited. The first objective was not difficult as the Bureau has designed and constructed many of these basins, some of which were checked with models. The principal task consisted of consulting laboratory records and tabulating the results. To accomplish the second objective required additional laboratory experiments.

ANALYSIS OF EXISTING DATA

Beginning with the first phase (1), the capacities and dimensions of 36 stilling basins for earth dams, small overflow dams and large canal structures, which have been tested by models, are listed in Table 2. The model studies were made in several laboratories by many individuals over a 23-year period. Each individual was more or less free to experiment with models of these structures as he saw fit. The final designs, tabulated in Table 2, represent an agreement between designer and experimenter for each case. Thus, the tabulation should be ideal for selecting a generalized design for Basin II.

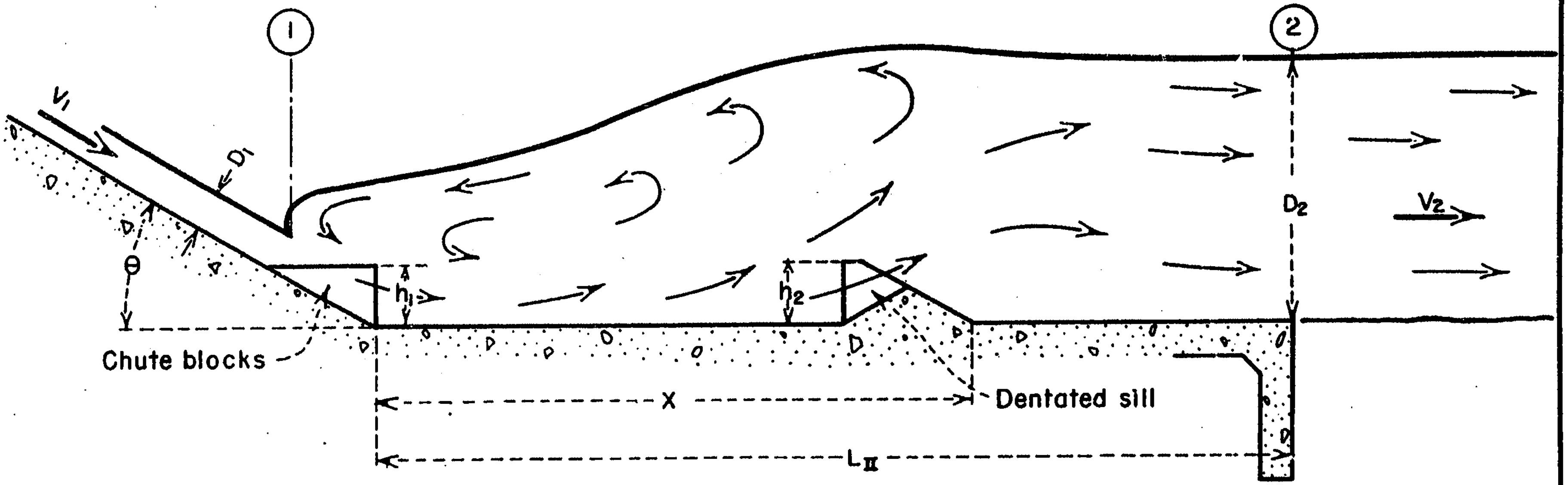


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Table 2

MODEL RESULTS ON EXISTING TYPE II STILLING BASINS

Dam	Report No (1)	Max reservoir el (2)	Tailwater el (3)	Fall headwater to tailwater ft (4)	Basin floor el (5)	Max discharge cfs (6)	W Width of basin ft (7)	q Discharge per ft of basin cfs (8)	V_1 (Actual) velocity ft/sec (9)	D_1 ft (10)	$F_1 = \frac{V_1}{\sqrt{gD_1}}$ (11)	Tw depth ft (12)	$\frac{\text{Tw depth}}{D_1}$ (13)	D_2 (Conjugate) ft (14)	$\frac{\text{Tw depth}}{D_2\ (\text{conjugate})}$ (15)	L Length of basin ft (16)	$\frac{L}{D_2\ (\text{conjugate})}$ (17)	Slope entering basin degrees with horizontal (18)
Rye Patch	Hyd-3.08	4123	4085.5	37.5	4062	20,000	110	182	*53	3.40	5.15	23.5	6.81	22.5	1.04	46	2.04	33
Unity	Hyd-211	3820	3771	49	3749	10,000	55.5	180	62	2.90	6.42	22	7.59	25.0	0.88	99	2.36	26.7
Alcova	Hyd-5.1	5500	5354	146	5309	55,000	150	367	98	3.75	8.88	43	12.00	45.5	0.97	125	2.75	25.0
Shadow Mountain	Hyd-162	8367	8332	35	8313	10,000	70	143	55	4.60	6.09	19	7.31	21.0	0.91	56	2.67	28
Boysen (Final)	Hyd-212	4758	4628	124	4594	20,000	66	303	85	3.55	7.98	34	9.58	38.2	0.89	131	3.95	34
Boysen (Prelim)	Hyd-212	4752	4646.5	105.5	4600	62,000	125	496	90	5.70	6.63	46.5	8.16	50.7	0.92	140	2.76	33.7
Scofield	Hyd-160	7630	7583.6	46.4	7564	6,200	40	155	61	2.54	6.74	19.6	7.72	23.0	0.85	60	2.61	33.7
Boca	Hyd-225	5605	5508	97	5487	8,000	75	107	73	1.47	10.53	21	14.28	21.2	0.99	58.6	2.66	26.7
Fresno	Hyd-177	2591	2528	63	2499.5	51,000	190	268	70	3.85	6.27	28.5	7.40	32.0	0.89	85	2.66	33.7
Bull Lake	Hyd-26.1	5805	5743.5	61.5	5725	10,000	100	100	59	1.70	7.97	18.5	10.88	18.3	1.01	75	4.10	14
Caballo	Hyd-72	4182	4118	64	4086	33,000	108	306	72	4.22	6.29	32	7.58	35.0	0.92	78.2	2.23	26.7
Moon Lake	Hyd-158	8137	8028.2	108.8	8005	10,000	75	133	74	1.80	9.74	23.2	12.90	23.8	0.98	60	2.52	26.7
Deer Creek	Hyd-215	5417	5285	132	5260	12,000	75	160	86	1.87	11.00	25	13.37	28.0	0.89	75	2.68	26.7
Alamogordo	Hyd-243	4275	4163	112	4118	56,000	110	509	96	5.30	7.33	45	8.49	58.0	0.87	125	2.40	26.7
Enders	Hyd-292	3129.5	3057	73	3016	200,000	400	500	79	6.33	5.54	41	6.38	46.5	0.88	115	2.47	26.7
Medicine Creek	Hyd-279	2408.9	2328	81	2287	97,800	262	373	80	6.32	5.55	41	7.31	47.0	0.87	125	2.66	26.7
Cedar Bluff	Hyd-330	2192	2074.3	118	2035.5	87,400	200	437	93	4.68	7.60	39	8.33	48.0	0.81	141	2.94	18.5
Falcon	Hyd-276	314.2	235	79	175	456,000	600	760	86	8.80	5.13	60	6.82	60.0	1.00	180	3.00	26.7
Trenton	Hyd-301	2783	2700.6	84	2653	133,000	266	500	81	6.20	5.76	48	7.74	47.6	1.01	125	2.63	18.5
Cachuma	Hyd-354	757.6	578.8	179	523	161,000	322	500	108	4.65	8.84	56	12.09	55.8	1.00	153	2.74	26.7
Tiber	Hyd-288	3014.9	2835.8	179	2797	54,250	200	271	97	2.79	10.26	39	13.98	39.0	1.00	117	3.25	18
Imperial Spillway	TM-516	191	168	23	150	150,000	1197.5	125	*49	2.60	5.44	16	6.16	18.5	0.87	41	2.22	14
Imperial Sluiceway	TM-516	181	155	26	140	24,000	248.3	97	*38	2.50	4.31	15	6.00	14.0	1.07	69	4.93	14
Grassy Lake	Hyd-226	7210	7100	110	7086	1,200	20	60	63	0.95	11.41	14	14.73	14.8	0.95	45	3.04	26.7
Box Butte	Hyd-94	4014	3961	53	3946.5	2,500	40	62	56	1.60	7.84	15	9.37	16.9	0.89	50	2.96	22
Siphon Drop Turnout	PVI-B4-BC	169.7	150.7	19	136	2,000	33.5	60	*40	1.50	5.80	15	10.0	11.5	1.30	36	3.13	22
Pilot Knob Wasteway	Hyd-184	170.26	124	46	94.5	13,155	140	94	*59	1.60	8.34	30	18.7	18.0	1.67	60	3.33	18.5
AA Canal Drop 1	PVI-B4-BC	43.6	29.2	14	13.3	8,700	118	74	*40	1.80	5.32	16	8.89	12.5	1.28	27	2.16	22
Wasteway No. 2	Hyd-108	1185.75	1027.4	159	1014.9	2,100	40	52	*92	0.60	22.00	12	21.4	16.5	0.73	45	2.73	24.7
Big Sandy No. 2	Hyd-121	6761.3	6702	59	6679	7,500	50	150	66	2.24	7.70	23	10.26	23.3	0.99	73	3.57	33.7
Cherry Creek	Hyd-146	5632.4	5558	74	5518	45,000	116	388	*80	4.90	6.46	40	8.16	42.0	0.95	120	2.86	25
Pine View	Hyd-3.04	4870	4817	53	4785	9,000	40	225	*59	3.80	5.42	32	8.42	27.0	1.19	96	3.55	33.7
Agency Valley	TM-506	3340	3266.5	74	3234	10,000	50	200	56	3.00	6.82	32	10.68	27.0	1.19	110	4.07	33.7
Davis		647	515.5	131	460	175,000	246	711	*97	7.30	6.41	56	7.64	62.0	0.90	100	1.61	14
Bonny	Hyd-331	3737.6	3623	114.6	3589	64,700	215	301	84	3.60	7.76	34	9.44	38.0	0.89	102	2.68	20
Cle Elum	TM-290	2240	2130	110	2097	40,000	200	200	*82	2.40	9.46	33	13.74	30.0	1.10	108	3.60	33.7
Maximum				179			1197.5	760	108	8.80	22.00	60	21.40	62.0	1.67	180	4.93	34
Minimum				14			20	32	38	0.60	4.31	12	6.00	11.5	0.73	27	1.61	14
Average				85				265							0.99		2.90	

*Estimated hydraulic losses.

Dam	Chute blocks: Type (19)	h_1 Height ft (20)	$\frac{h_1}{D_1}$ (21)	w_1 Width ft (22)	$\frac{w_1}{h_1}$ (23)	$\frac{s_1}{w_1}$ (24)	End sill: Type (25)	Position on apron $\frac{X}{L_B}$ (26)	h_2 Height ft (27)	$\frac{h_2}{D_2}$ (28)	w_2 Width ft (29)	$\frac{w_2}{h_2}$ (30)	$\frac{s_2}{w_2}$ (31)	Intermediate baffle blocks: Type (32)	Position on apron $\frac{X}{L_B}$ (33)	Height ft (34)	$\frac{\text{Height}}{D_2}$ (35)	Width ft (36)	$\frac{\text{Width}}{\text{Height}}$ (37)	$\frac{\text{Width}}{\text{Spacing}}$ (38)	Type of basin (39)	Wing walls at end of basin (40)
Rye Patch	Solid	4.0	1.18	--	--	--	Solid	1.0	2.0	0.09	--	--	--	--	--	--	--	--	--	--	Rectangular	45° warped
Unity	Teeth	3.5	1.21	1.83	0.52	1.0	Teeth	0.93	5.5	0.22	2.17	0.39	1.0	--	--	--	--	--	--	--	R	Normal
Alcova	Solid	4.3	1.15	--	--	--	T	1.0	10.0	0.22	5.0	0.50	1.0	--	--	--	--	--	--	--	R	None
Shadow Mountain	T	3.0	1.16	4.0	1.33	1.78	T	1.0	3.5	0.17	4.0	1.14	1.78	--	--	--	--	--	--	--	R	Normal
Boysen (Final)	T	4.0	1.13	5.25	1.31	1.75	T	0.68	8.75	0.23	5.25	0.60	1.75	--	--	--	--	--	--	--	R	None
Boysen (Prelim)	T	6.0	1.05	7.5	1.25	1.8	T	1.0	8.0	0.16	7.5	0.94	1.8	--	--	--	--	--	--	--	R	None
Scofield	T	3.5	1.38	3.5	1.0	1.91	T	1.0	4.0	0.17	3.5	0.88	1.91	--	--	--	--	--	--	--	R	Normal
Boca	T	4.0	2.72	3.0	0.75	1.0	T	1.0	4.0	0.19	3.0	0.75	1.0	--	--	--	--	--	--	--	R	Normal
Fresno	T	4.0	1.04	4.0	1.0	1.0	T	1.0	6.0	0.19	4.0	0.67	1.0	--	--	--	--	--	--	--	R	45° warped
Bull Lake	None	--	--	--	--	--	T	1.0	4.0	0.22	5.0	1.25	1.0	--	--	--	--	--	--	--	R	Normal
Caballo	T	4.5	1.07	4.0	0.89	1.0	T	0.90	6.5	0.19	4.0	0.62	1.0	--	--	--	--	--	--	--	R	Normal
Moon Lake	T	2.6	1.44	1.875	0.72	1.0	T	0.85	5.0	0.21	3.7*	0.75	1.0	--	--	--	--	--	--	--	R	Normal
Deer Creek	T	3.0	1.60	3.0	1.0	1.0	T	1.0	5.0	0.18	3.0	0.60	1.0	--	--	--	--	--	--	--	R	Normal
Alamogordo	T	8.0	1.51	3.5	0.44	1.0	T	1.0	9.0	0.17	4.0	0.44	1.0	--	--	--	--	--	--	--	R	Normal
Enders	T	6.0	0.95	5.0	0.83	1.0	T	1.0	12.0	0.26	5.0	0.42	1.0	--	--	--	--	--	--	--	R	Normal
Medicine Creek	T	6.75	1.07	6.0	0.89	1.0	T	1.0	8.0	0.17	6.0	0.75	1.0	--	--	--	--	--	--	--	R	Normal
Cedar Bluff	T	7.0	1.49	6.0	0.86	1.0	T	1.0	9.0	0.19	6.0	0.67	1.0	--	--	--	--	--	--	--	R	Normal
Falcon	T	8.0	0.91	10.0	1.25	1.0	T	1.0	12.0	0.20	10.0	0.83	1.0	--	--	--	--	--	--	--	R	45° vertical
Trenton	T	5.0	0.81	5.0	1.0	1.0	T	1.0	9.0	0.19	5.0	0.56	1.0	--	--	--	--	--	--	--	R	Normal
Cachuma	T	5.5	1.19	5.08	0.92	1.0	T	1.0	12.0	0.21	5.08	0.42	1.0	--	--	--	--	--	--	--	R	Normal
Tiber	T	7.0	2.51	5.0	0.71	1.0	T	1.0	8.0	0.21	5.0	0.62	1.0	--	--	--	--	--	--	--	R	Normal
Imperial Spillway	T	2.33	0.90	2.49	1.02	1.03	T	1.0	3.75	0.20	3.87	1.01	1.0	--	--	--	--	--	--	--	R	20° warped
Imperial Sluiceway	T	3.33	1.33	3.0	0.90	0.95	T	0.65	5.0	0.36	5.67	1.13	1.1	--	--	--	--	--	--	--	R	20° warped
Grassy Lake	T	1.0	1.05	1.67	1.67	1.0	T	1.0	2.0	0.14	1.33	0.66	1.0	--	--	--	--	--	--	--	R	Normal
Box Butte	T	3.3	2.06	3.0	0.91	1.0	T	1.0	3.0	0.18	3.0	1.0	1.0	--	--	--	--	--	--	--	R	Normal
Siphon Drop Turnout	T	2.25	1.50	2.25	1.0	1.0	T	1.0	3.25	0.29	2.25	0.69	1.0	T	0.72	3.25	0.08	2.25	1.44	1.0	R	45° warped
Pilot Knob Wasteway	T	2.5	1.56	2.5	1.0	1.0	T	1.0	5.0	0.28	5.0	1.0	1.0	--	--	--	--	--	--	--	R	45° warped
AA Canal Drop 1	T	1.75	0.97	1.75	1.0	1.0	Solid	1.0	1.0	0.08	--	--	--	--	--	--	--	--	--	--	R	30° warped
Wasteway No. 2	Vanes	--	--	--	--	--	Solid	1.0	2.0	0.12	--	--	--	T	.71	2.41	0.15	2.41	1.0	1.0	R	30° warped
Big Sandy No. 2	T	2.5	1.12	3.25	1.30	1.0	T	1.0	3.0	0.12	3.25	1.08	1.0	--	--	--	--	--	--	--	Trapezoidal	Normal
Cherry Creek	None	--	--	--	--	--	Solid	1.0	5.0	0.12	--	--	--	T	.73	5.0	0.12	3.75	0.75	1.0	R	None
Pine View	None	--	--	--	--	--	T	1.0	5.0	0.19	6.0	1.20	1.0	--	--	--	--	--	--	--	T	Warped curve
Agency Valley	Solid	2.5	0.83	--	--	--	T	0.91	10.0	0.37	3.33	0.33	1.0	--	--	--	--	--	--	--	T	30° warped
Davis	T	14.3	1.96	13.0	0.91	1.86	T	1.0	14.3	0.23	13.0	0.91	1.86	--	--	--	--	--	--	--	R	Normal
Bonny	T	7.0	1.94	5.0	0.72	1.0	T	1.0	8.0	0.21	5.0	0.62	1.0	--	--	--	--	--	--	--	R	Normal
Cle Elum	None	--	--	--	--	--	T	1.0	10.0	0.33	10.0	1.0	1.0	--	--	--	--	--	--	--	R	None
Maximum			2.72		1.67	1.91		1.0		0.37		1.25	1.91		.73		0.28		1.44	1.0		
Minimum			0.81		0.44	0.95		0.65		0.08		0.33	1.00		.71		0.12		0.75	1.0		
Average			1.35		0.97	1.15		0.97		0.20		0.76	1.13							1.0		

Results of Compilation

With the aid of Figure 10, most of the symbols used in Table 2 are self-explanatory. The use of baffle piers is limited to Basin III. Column 1 lists the reference material used in compiling the table. Column 2 lists the maximum reservoir elevation, Column 3 the maximum tail water elevation, Column 5 the elevation of the stilling basin floor, and Column 6 the maximum discharge for each spillway. Column 4 indicates the height of the structure studied, showing a maximum fall from headwater to tail water of 179 feet, a minimum of 14 feet, and an average of 85 feet. Column 7 shows that the width of the stilling basins varied from 1,197.5 to 20 feet. The discharge per foot of basin width, Column 8, varied from 760 to 52 cfs, with 265 as an average. The computed velocity, V_1, (hydraulic losses estimated in some cases), entering the stilling basin (Column 9) varied from 108 to 38 feet per second, and the depth of flow, D_1, entering the basin (Column 10) varied from 8.80 to 0.60 feet. The value of the Froude number (Column 11) varied from 22.00 to 4.31. Column 12 shows the actual depth of tail water above the stilling basin floor, which varied from 60 to 12 feet, while Column 14 lists the computed, or conjugate, tail water depth for each stilling basin. The conjugate depths, D_2, were obtained from Figure 5. The ratio of the actual tail water depth to the conjugate depth is listed for each basin in Column 15.

Tail water depth. The ratio of actual tail water depth to conjugate depth shows a maximum of 1.67, a minimum of 0.73, and an average of 0.99. In other words, on the average, the basin floor was set to provide a tail water depth equal to the conjugate or necessary depth.

Chute blocks. The chute blocks used at the entrance to the stilling basin varied in size and spacing. Some basins contained nothing at this point, others a solid step, but in the majority of cases a serrated device, known as chute blocks, was utilized. The chute blocks at the upstream end of the basin tend to corrugate the jet, lifting a portion of it from the floor, resulting in a shorter length of jump than would be possible without them. These blocks also tend to improve the action in the jump. The proportioning of chute blocks has been the subject of much discussion. The tabulation in Columns 19 through 24 of Table 2 shows the sizes which have been used. Column 20 shows the height of the chute blocks, while Column 21 gives the ratio of height of block to the depth D_1. The ratios of height of block to D_1 indicate a maximum of 2.72, a minimum of 0.81, and an average of 1.35. This is somewhat higher than was shown to be necessary by the verification tests discussed later; a block equal to D_1 in height is sufficient.

The width of the blocks is shown in Column 22. Column 23 gives the ratio of width of the block to height, with a maximum of 1.67, a minimum of 0.44, and an average of 0.97. The ratio of width of block to spacing, tabulated in Column 24, shows a maximum of 1.91, a minimum of 0.95, and an average of 1.15. The three ratios indicate that the proportion: height equals width, equals spacing, equals D_1 should be a satisfactory standard for chute block design. The wide variation shows that these dimensions are not critical.

Dentated sill. The sill in or at the end of the basin was either solid or had some form of dentated arrangement, as designated in Column 25. A dentated sill located at the end of the apron is recommended. The shape of the dentates and the angle of the sills varied considerably in the spillways tested, Columns 26 through 31. The position of the dentated sill also varied and this is indicated by the ratio $\frac{X}{L_{II}}$ in Column 26. The distance, X, is measured to the downstream edge of the sill, as illustrated in Figure 10. The ratio $\frac{X}{L_{II}}$ varied from 1 to 0.65, with an average of 0.97.

The heights of the dentates are given in Column 27. The ratio of height of block to the conjugate tail water depth is shown in Column 28. These ratios show a maximum of 0.37, a minimum of 0.08, and an average of 0.20. The width to height ratio, Column 30, shows a maximum of 1.25, a minimum of 0.33, and an average of 0.76. The ratio of width of block to spacing, Column 31, shows a maximum of 1.91, a minimum of 1.0, and an average of 1.13. For the sake of generalization, the following proportions are recommended: (1) height of dentated sill = $0.2D_2$, (2) width of blocks = $0.15D_2$, and (3) spacing of blocks = $0.15D_2$, where D_2 is the conjugate tail water depth. It is recommended that the dentated sill be placed at the downstream end of the apron.

Columns 32 through 38 show the proportions of additional baffle blocks used on three of the stilling basins. These are not necessary and are not recommended for this type of basin.

Additional details. Column 18 indicates the angle, with the horizontal, at which the high-velocity jet enters the stilling basin for each of the spillways. The maximum angle was 34° and the minimum 14°. The effect of the vertical angle of the chute on the action of the hydraulic jump could not be evaluated from the information available. This factor will be considered, however, in Section 5 in connection with sloping apron design.

Column 39 designates the cross section of the basin. In all but three cases the basins were rectangular. The three cross sections that were trapezoidal had side slopes varying from 1/4:1 to 1/2:1. The generalized designs presented in this report are for stilling basins with rectangular cross sections. Where trapezoidal basins are used, a model study is strongly recommended.

Designers have been concerned over the type of wing wall which should be used at the end of stilling basins. Column 40, Table 2, indicates that in the majority of basins constructed for earth dam spillways the wing walls were normal to the training walls. Five basins were constructed without wing walls using a rock blanket for protection. The remainder utilized angling wing walls or warped transitions downstream from the basin. The latter are common on canal structures. The object, of course, is to build the cheapest wing wall that will afford the necessary protection. The type of wing wall is usually dictated by local conditions such as width of the channel downstream, depth to foundation rock, degree of protection needed, etc., thus wing walls are not amenable to generalization.

VERIFICATION TESTS

It was early learned that the information on Table 2 did not cover the entire range of operating conditions desired. There was insufficient information to determine the length of basin for the larger values of the Froude number; there was little or no information on the tail water depth at which sweepout occurs, and the information available was of little value for generalizing water-surface profiles. It was, therefore, necessary to perform a set of experiments to extend the range and to supply the missing data. The experiments were made on 17 Type II basins, proportioned according to the above rules, and installed in Flumes B, C, D, and E (see Columns 1 and 2, Table 3). Each basin was judged at the discharge for which it was designed; the length was adjusted to the minimum that would produce satisfactory operation, and the absolute minimum tail water depth for acceptable operation was measured. The basin operation was also observed for flows less than the designed discharge and found to be satisfactory in each case.

Table 3 is quite similar to Table 2 with the exception that the length of basin L_{II} (Column 11) was determined by experiment, and the tail water depth at which the jump just began to sweep out of the basin was recorded (Column 13).

Tail Water Depth

The solid line on Figure 11 was obtained from the hydraulic jump formula $\frac{D_2}{D_1} = 1/2\ (\sqrt{1+8F^2}-1)$ and represents conjugate tail water depth. It is the same as the line shown on Figure 5. The dash lines on Figure 11 are merely guides drawn for tail water depths other than conjugate depth. The points shown as dots were obtained from Column 13 of Table 2 and constitute the ratio of actual tail water depth to D_1 for each basin listed. It can be observed that the majority of the basins were designed for conjugate tail water depth or less. The minimum tail water depth for Basin II, obtained from Column 14 of Table 3, is shown on Figure 11. The curve labeled "Minimum TW Depth Basin II" indicates the point at which the front of the jump moves away from the chute blocks. In other words, any additional lowering of the tail water would cause the jump to leave the basin. Consulting Figure 11 it can be observed that the margin of safety for a Froude number of 2 is 0 percent; while for a number of 6 it increases to 6 percent, for a number of 10 it diminishes to 4 percent, and for a number of 16 it is 2.5 percent. To be certain that this is understood, it will be stated another way. The jump will no longer operate properly when the tail water depth approaches $0.98D_2$ for a Froude number of 2 or $0.94D_2$ for a number of 6 or $0.96D_2$ for a number of 10, or $0.975D_2$ for a number of 16. The margin of safety is largest in the middle range. For the two extremes of the curve it is advisable to provide a tail water greater than conjugate depth to be safe. For these reasons the Type II basin should never be designed for less than conjugate depth and a minimum safety factor of 5 percent of D_2 is recommended.

There are several other considerations in regard to tail water which are mentioned as a reminder. First, tail water curves are usually extrapolated for the discharges encountered in design, so they can be in error. Secondly, the actual tail water depth usually lags, in a temporal sense, that of the tail water curve for rising flow and leads the curve for a falling discharge. Extra tail water should therefore be provided if reasonable increasing increments of discharge limit the performance of the structure because of a lag in building up tail water depth. Thirdly, a tail water curve may be such that the most adverse condition occurs at less than the maximum designed discharge; and fourthly, temporary or permanent retrogression of the riverbed downstream may be a factor needing consideration. These factors, some of which are difficult to evaluate, are all important in stilling basin design, and suggest that an adequate factor of safety is essential. It is advisable to construct a jump height curve, superimposed on the tail water curve, for each basin to determine the most adverse operating condition. This procedure will be illustrated later.

Table 3

VERIFICATION TESTS ON TYPE II STILLING BASIN

Flume	Test	Q Total discharge cfs	W Width of basin ft	q Discharge per foot of basin width cfs	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_{II} Length of jump ft	$\frac{L_{II}}{D_2}$	T_{so} Tailwater at sweep out ft	$\frac{T_{so}}{D_1}$	$\frac{T_{so}}{D_2}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
B	1	2.800	2.000	1.250	1.120	17.36	0.072	15.60	11.39	4.96	4.42	1.09	15.13	0.97
	2	4.000		2.000	1.430	17.54	0.114	12.54	9.16	6.10	4.27	1.37	12.02	0.96
	3	6.000		3.000	1.750	17.65	0.170	10.29	7.54	7.30	4.17	1.65	9.70	0.94
	4	8.000		4.000	2.030	17.86	0.224	9.06	6.64	8.00	3.94	1.88	8.39	0.93
C	5	1.600	1.500	1.067	1.070	17.49	0.061	17.54	12.48	4.60	4.30	1.04	17.04	0.97
	6	2.100		1.400	1.240	17.94	0.078	15.89	11.32	5.40	4.35	1.18	15.12	0.95
	7	2.630		1.753	1.355	18.26	0.096	14.11	10.39	5.70	4.21	1.32	13.75	0.97
	8	2.750		1.833	1.400	18.33	0.100	14.00	10.21	6.23	4.45	1.36	13.60	0.97
	9	4.000		2.667	1.785	20.36	0.131	13.62	9.91	7.40	4.15	1.73	13.21	0.97
D	10	5.000	3.970	1.259	1.235	20.30	0.062	19.91	14.38	5.10	4.13	1.20	19.35	0.97
	11	6.000		1.511	1.350	20.41	0.074	18.24	13.21	5.80	4.30	1.32	17.83	0.98
	12	9.800		2.468	1.750	21.84	0.113	15.50	11.45	7.80	4.46	1.73	15.31	0.99
	13	11.000		2.770	1.855	21.15	0.131	14.16	10.29	8.10	4.37	1.82	13.89	0.98
	14	13.000		3.273	2.020	21.39	0.153	13.20	9.64	8.70	4.31	1.95	12.75	0.97
	15	20.000		5.036	2.585	23.00	0.319	11.80	8.66	10.60	4.10	2.48	11.32	0.96
E	16	5.000	3.970	1.259	0.840	10.49	0.120	7.00	5.33	3.36	4.00	0.79	6.58	0.94
	17	10.000		2.518	1.220	11.09	0.227	5.37	4.10	4.51	3.70	1.10	9.02	0.90

Flume	Test	h_1 Height chute blocks ft	$\frac{h_1}{D_1}$	$\frac{w_1}{h_1}$	$\frac{s_1}{h_1}$	h_2 Height dentated sill ft	$\frac{h_2}{D_2}$	$\frac{w_2}{h_2}$	$\frac{s_2}{h_2}$	α Slope water surface degrees	θ Slope of Chute
		(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
B	1	0.073	1.01	1.0	1.0	0.219	0.196	0.75	0.75	10.5	0.7:1
	2	0.114	1.00	1.0	1.0	0.286	0.200	0.75	0.75	10.0	
	3	0.170	1.00	1.0	1.0	0.352	0.201	0.75	0.75	9.6	
	4	0.229	1.02	1.0	1.0	0.406	0.200	0.75	0.75	9.0	
C	5	0.062	1.02	1.0	1.0	0.320	0.300	0.75	0.75	11.3	2:1
	6	0.078	1.00	1.0	1.0	0.260	0.210	0.75	0.75	10.8	
	7	0.105	1.09	1.0	1.0	0.250	0.185	0.75	0.75	10.5	
	8	0.100	1.00	1.0	1.0	0.310	0.221	0.75	0.75	10.0	
	9	0.131	1.00	1.0	1.0	0.446	0.250	0.75	0.75	10.4	
D	10	0.062	1.00	1.0	1.0	0.250	0.203	1.00	1.00	12.0	0.6:1
	11	0.074	1.00	1.0	1.0	0.270	0.200	1.00	1.00	11.2	
	12	0.153	1.35	1.0	1.0	0.400	0.229	1.00	1.00	10.0	
	13	0.131	1.00	1.0	1.0	0.396	0.214	0.75	0.75	10.2	
	14	0.153	1.00	1.0	1.0	0.400	0.198	0.75	0.75	8.3	
	15	0.219	1.00	1.0	1.0	0.517	0.200	0.75	0.75	9.5	
E	16	0.122	1.02	1.0	1.0	0.200	0.238	0.75	0.75	6.5	Varied
	17	0.235	1.04	1.0	1.0	0.270	0.221	0.75	0.75	5.3	

The verification tests repeatedly demonstrated that there is no simple remedy for a deficiency in tail water depth. Increasing the length of basin, which is the remedy often attempted in the field, will not compensate for deficiency in tail water depth. For these reasons, care should be taken to consider all factors that may affect the tail water at a future date. A stilling basin that does not perform properly cannot be justified in the light of money saved by skimping, regardless of the amount.

Length of Basin

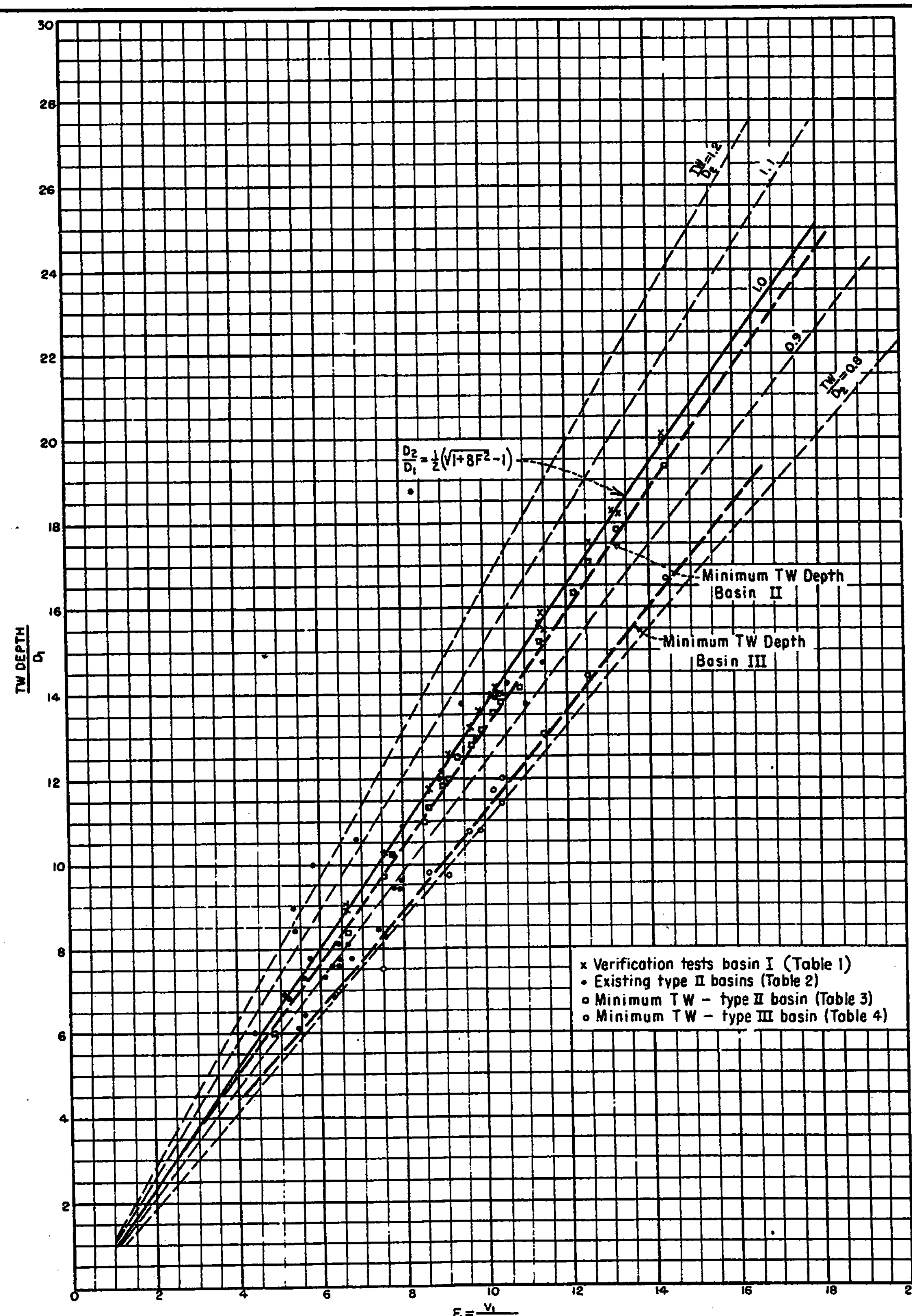
The necessary length of Basin II, determined by the verification tests, is shown as the intermediate curve on Figure 12. The squares indicate the test points (Columns 10 and 12 of Table 3). The black dots represent existing basins (Columns 11 and 17, Table 2). Conjugate depth was used in the ordinate ratio rather than actual tail water depth since it could be computed for each case.

The dots scatter considerably but an average curve drawn through these points would be lower than the Basin II curve. In Figure 12, therefore, it appears that in practice a basin about 3 times the conjugate depth is actually used when a basin about 4 times the conjugate is recommended from the verification tests. It should be remembered, however, that the shorter basins were all model tested and every opportunity was taken to reduce the basin length. The extent and depth of bed erosion, wave heights, favorable flood frequencies, flood duration and other factors were all used to justify reducing the basin length. Lacking definite knowledge of this type in designing a basin for field construction without model tests, the longer basins indicated by the verification tests curve are recommended.

The Type II basin curve has been arbitrarily terminated at Froude number 4, as the jump may be unstable at lower numbers. The chute blocks have a tendency to stabilize the jump and reduce the 4.5 limit discussed for Basin I. For basins having Froude numbers below 4.5 see Section 4.

Water-surface Profiles

Water-surface profiles were measured during the tests to aid in computing uplift pressures under the basin apron. As the water surface in the stilling basin tests fluctuated rapidly it was felt that a high degree of accuracy in measurement was not necessary. This was found to be true when the approximate water-surface profiles obtained were plotted, then generalized. It was found that the profile in the basin could be closely approximated by a straight line making an angle α with the horizontal. This line can also be considered to be a pressure profile.



HYDRAULIC JUMP STUDIES
STILLING BASINS I, II AND III
MINIMUM TAILWATER DEPTHS

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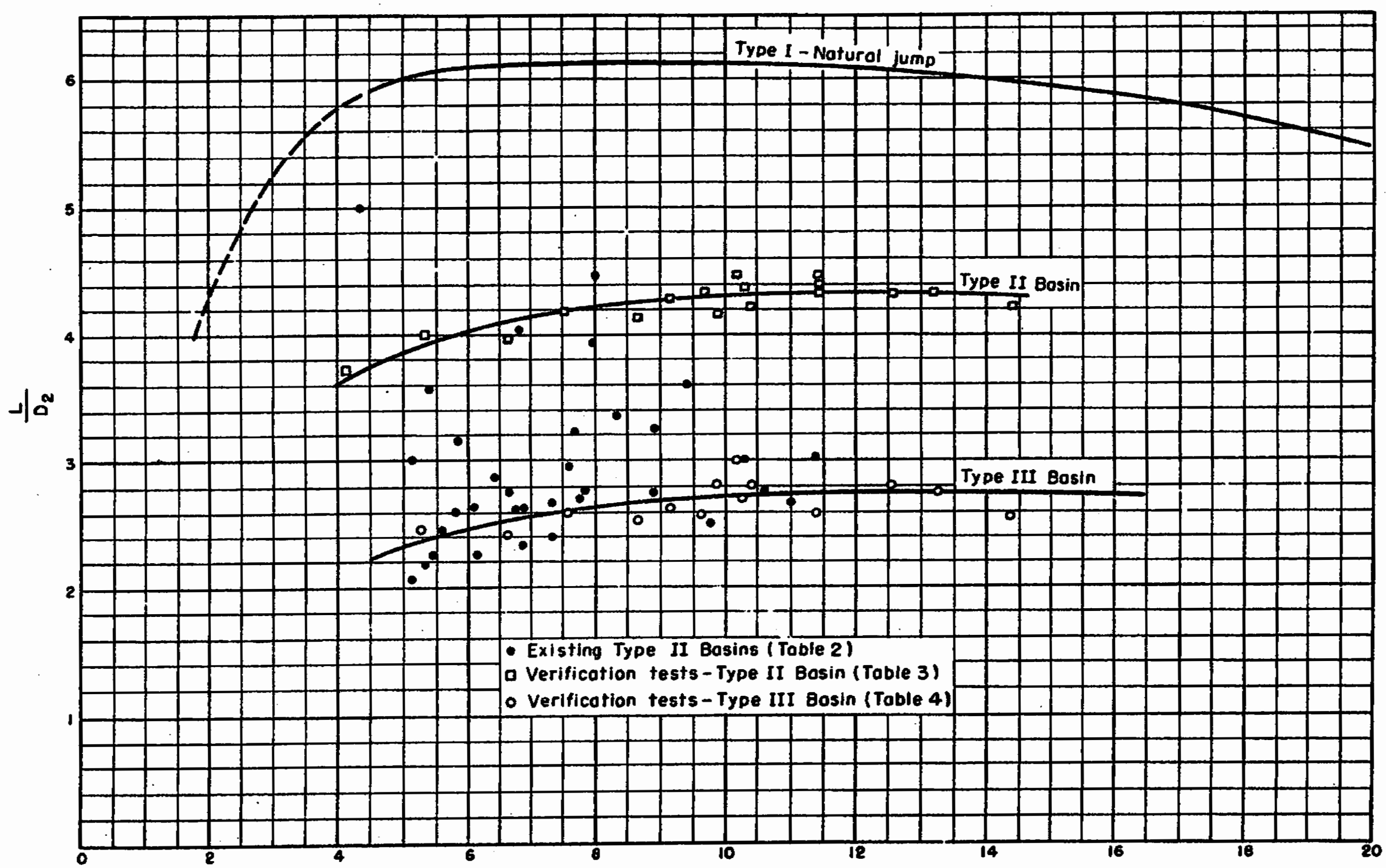
FIGURE 12



$$F_1 = \frac{V_1}{\sqrt{g D_1}}$$

HYDRAULIC JUMP STUDIES

STILLING BASINS I, II, AND III

LENGTH OF JUMP ON HORIZONTAL FLOOR

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The angle α (Column 24, Table 3) observed in each of the verification tests has been plotted with respect to the Froude number on Figure 13. The slope increases with the Froude number. To use the curve in Figure 13, a horizontal line is drawn at conjugate depth on a scale drawing of the basin. A vertical line is also drawn from the upstream face of the dentated sill. Beginning at the point of intersection, a sloping line is constructed as shown. The above procedure gives the approximate water surface and pressure profile for conjugate tail water depth. Should the tail water depth be greater than D_2, the profile will resemble the uppermost line on Figure 13; the angle remains unchanged. This information applies only for the Type II basin, constructed as recommended in this section.

CONCLUSIONS

The following rules are recommended for generalization of Basin II, Figure 14:

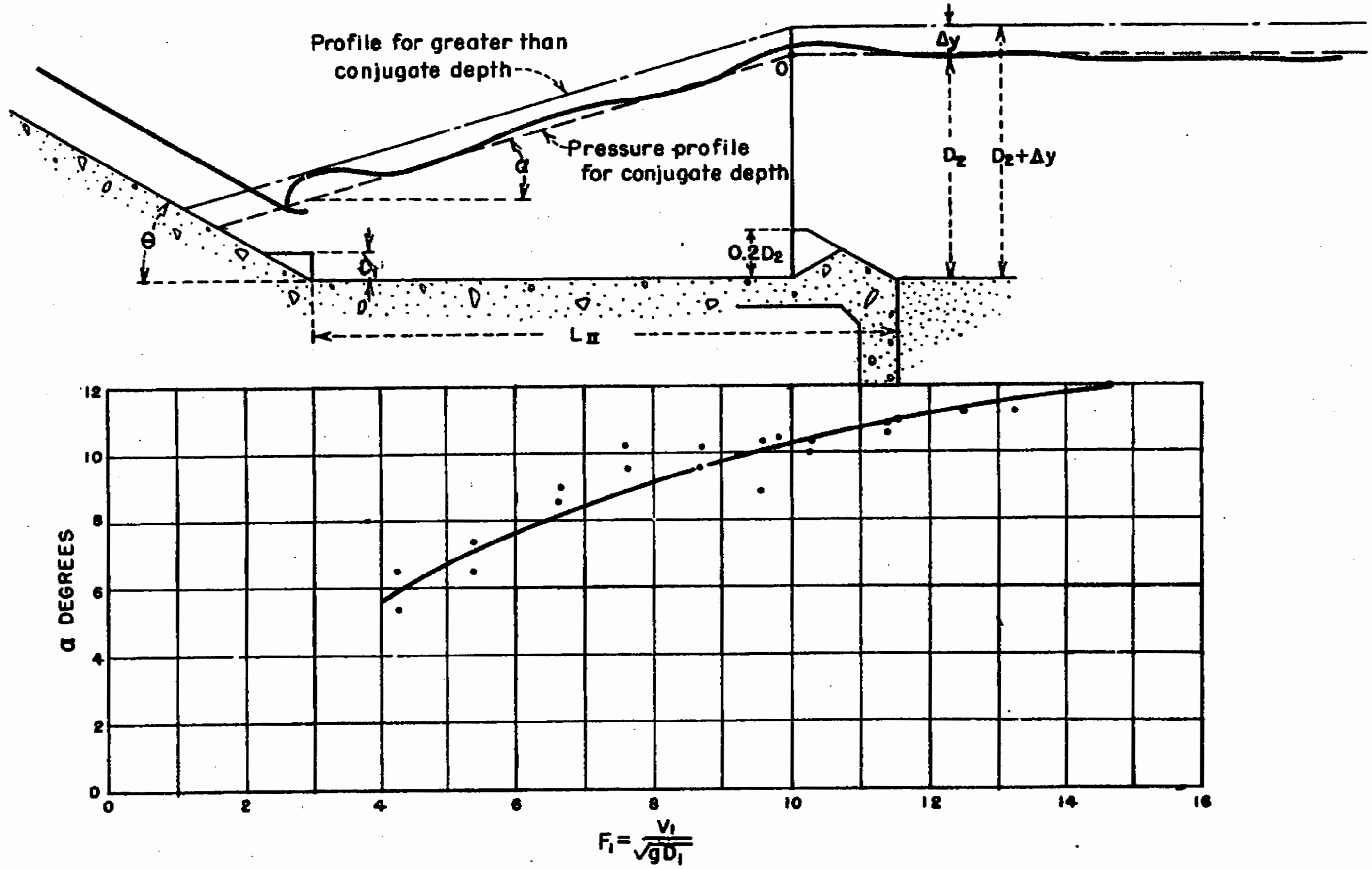
1. Set apron elevation to utilize full conjugate tail water depth, plus an added factor of safety if needed. An additional factor of safety is advisable for both low and high values of the Froude number (see Figure 11). A minimum margin of safety of 5 percent of D_2 is recommended.

2. Basin II may be effective down to a Froude number of 4 but the lower values should not be taken for granted (see Section 4 for values less than 4.5).

3. The length of basin can be obtained from the intermediate curve on Figure 12.

4. The height of chute blocks is equal to the depth of flow entering the basin, or D_1, Figure 14. The width and spacing should be equal to approximately D_1; however, this may be varied to eliminate the need of fractional blocks. A space equal to $\frac{D_1}{2}$ is preferable along each wall to reduce spray and maintain desirable pressures.

5. The height of the dentated sill is equal to $0.2D_2$, while the maximum width and spacing recommended is approximately $0.15D_2$. In this case a block is recommended adjacent to each side wall, Figure 14. The slope of the continuous portion of the end sill is 2:1. In the case of narrow basins, which would involve only a few dentates according to the above rule, it is advisable to reduce the width and the spacing so long as this is done proportionately. Reducing the width and spacing actually improves the performance in narrow basins, thus, the minimum width and spacing of the dentates is governed only by structural considerations.



HYDRAULIC JUMP STUDIES
STILLING BASIN II
APPROXIMATE WATER SURFACE AND PRESSURE PROFILES

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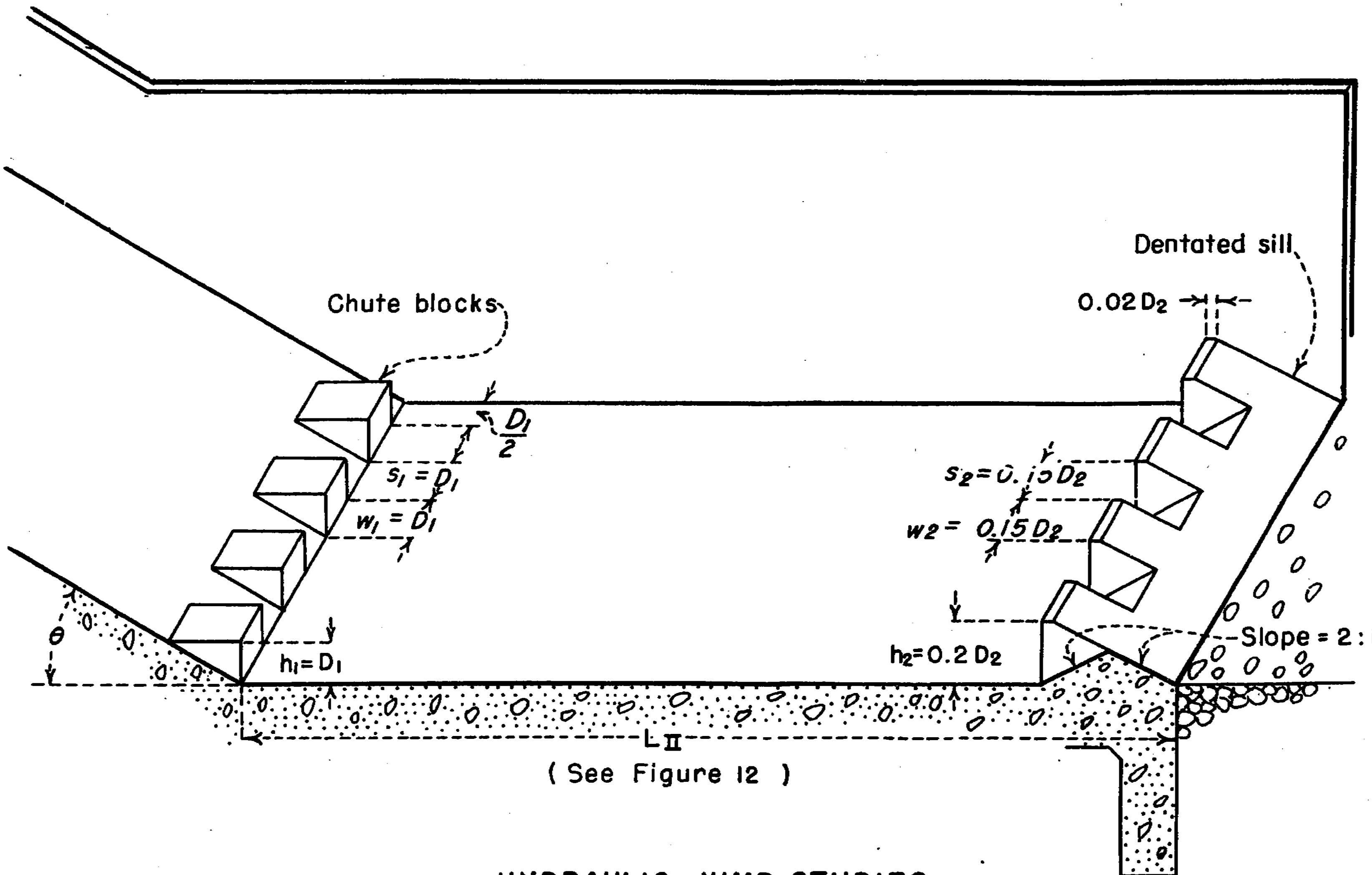
FIGURE 14



(See Figure 12)

HYDRAULIC JUMP STUDIES
STILLING BASIN II
RECOMMENDED PROPORTIONS

6. It is not necessary to stagger the chute blocks and the sill dentates. In fact this practice is usually inadvisable from a construction standpoint.

7. The verification tests on Basin II indicated no perceptible change in the stilling basin action with respect to the slope of the chute preceding the basin. The slope of chute varied from 0.6:1 to 2:1 in these tests, Column 25, Table 3. Actually, the slope of the chute does have an effect on the hydraulic jump in some cases. This subject will be discussed in more detail in Section 5 with regard to sloping aprons. It is recommended that the sharp intersection between chute and basin apron, Figure 14, be replaced with a curve of reasonable radius ($R \gtreqless 4D_1$) when the slope of the chute is 1:1 or greater. Chute blocks can be incorporated on the curved face as readily as on the plane surfaces.

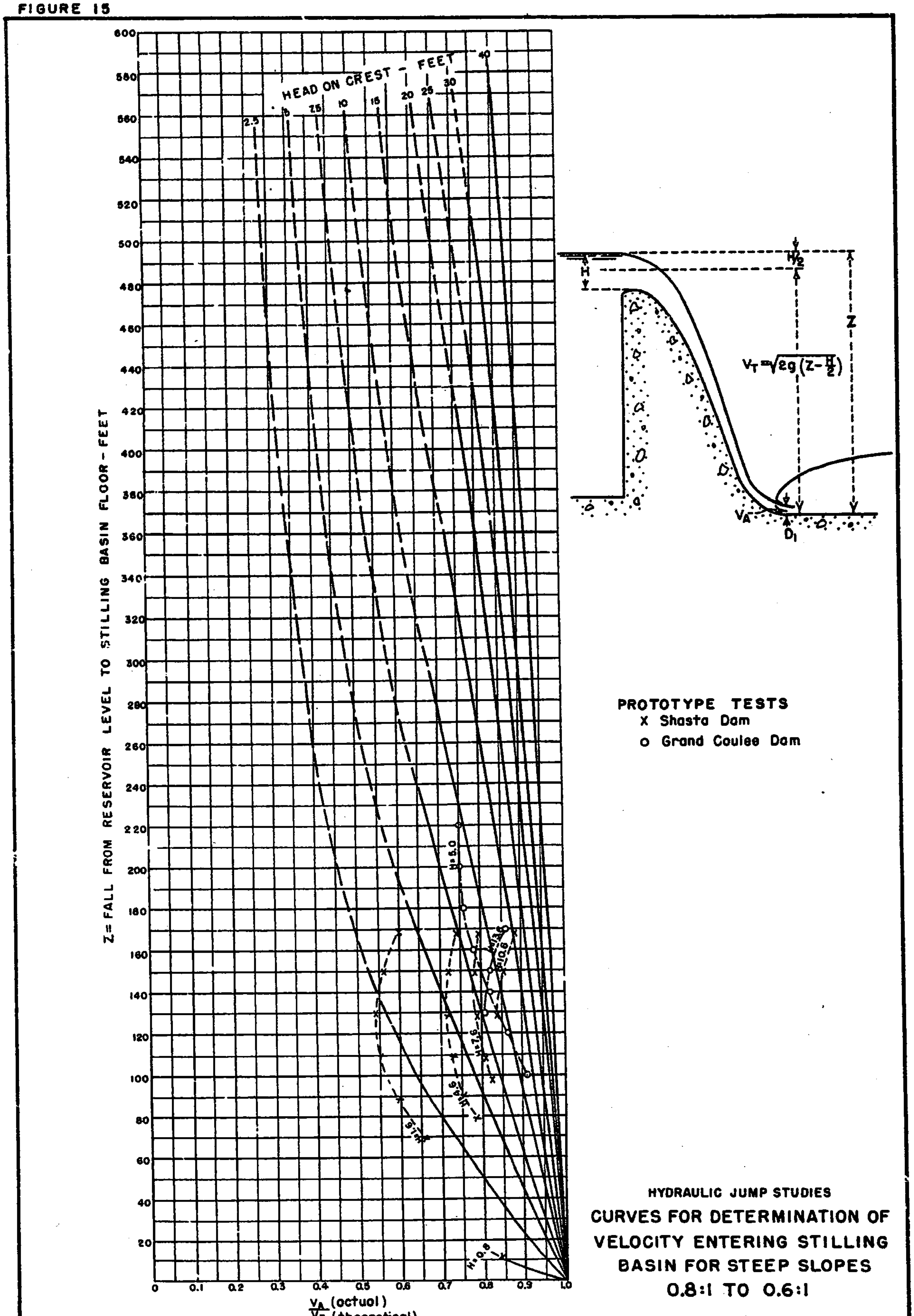
Following the above rules will result in a safe, conservative stilling basin for spillways with fall up to 200 feet and for flows up to 500 cfs per foot of basin width, providing the jet entering the basin is reasonably uniform both as to velocity and depth. For greater falls, larger unit discharges, or possible asymmetry, a model study of the specific design is recommended.

Aids in Computation

Previous to presenting an example illustrating the method of proportioning Basin II, a chart will be presented which should be of special value for preliminary computations. The chart makes it possible to determine V_1 and D_1 with a fair degree of accuracy, for chutes having slopes of 0.8:1 or steeper, where computation is a difficult and arduous procedure. The chart presented as Figure 15 represents a composite of experience, computation, and a limited amount of experimental information obtained from prototype tests on Shasta and Grand Coulee Dams. There is much to be desired in the way of experimental confirmation; however, it is felt that this chart is sufficiently accurate for preliminary design. A concerted effort will be made to obtain additional experimental information whenever possible.

The ordinate on Figure 15 is fall from reservoir level to stilling basin floor, while the abscissa is the ratio of actual to theoretical velocity at entrance to the stilling basin. The theoretical velocity $V_T = \sqrt{2g(Z-H/2)}$ (see Figure 15). The actual velocity is the term desired. The curves represent different heads, H, on the crest of the spillway. As is reasonable, the larger the head on the crest, the more nearly the actual velocity at the base of the spillway will approach the theoretical. For example, with H = 40 feet and Z = 230 feet, the actual velocity at the base of the dam would be 0.95 of the computed theoretical velocity; while with a head of 10 feet on the crest, the actual velocity would be 0.75 V_T. The value of D_1 is computed by dividing the unit discharge by the actual velocity obtained from Figure 15.

FIGURE 15



HYDRAULIC JUMP STUDIES

CURVES FOR DETERMINATION OF VELOCITY ENTERING STILLING BASIN FOR STEEP SLOPES 0.8:1 TO 0.6:1

The chart is not applicable for chutes flatter than 0.6:1 as frictional resistance assumes added importance in this range. Therefore, it will be necessary to compute the draw-down curve as usual starting at the gate section where critical depth is known.

Insufflation, produced by air from the atmosphere mixing with the sheet of water during the fall, need not be considered in the hydraulic jump computations. Insufflation need be considered principally in the design of chute and stilling basin walls. It is not possible to construct walls sufficiently high to confine all spray and splash; thus, the best that can be hoped for is a height that is reasonable and commensurate with the material and terrain to be protected.

Application of Results (Example 2)

The crest of an overfall dam, having a downstream slope of 0.7:1, is 200 feet above the horizontal floor of the stilling basin. The head on the crest is 30 feet and the maximum discharge is 480 cfs per foot of stilling basin width. Proportion a Type II stilling basin for these conditions.

Entering Figure 15 with a head of 30 feet over the crest and a total fall of 230 feet,

$$\frac{V_A}{V_T} = 0.92$$

The theoretical velocity $V_T = \sqrt{2g(230-\frac{30}{2})} = 117.6$ ft/sec

The actual velocity $V_A = V_1 = 117.6 \times 0.92 = 108.2$ ft/sec

$$D_1 = \frac{q}{V_1} = \frac{480}{108.2} = 4.44 \text{ feet}$$

The Froude number

$$F_1 = \frac{V_1}{\sqrt{gD_1}} = \frac{108.2}{\sqrt{32.2 \times 4.44}} = 9.04$$

Entering Figure 11 with a Froude number of 9.04, the solid line gives,

$$\frac{TW}{D_1} = 12.3$$

As TW and D_2 are synonomous in this case, the conjugate tail water depth,

$$D_2 = 12.3 \times 4.44 = 54.6 \text{ feet}$$

The minimum tail water line for the Type II basin on Figure 11 shows that a factor of safety of about 4 percent can be expected for the above Froude number.

Should it be desired to provide a margin of safety of 7 percent, the following procedure may be followed: Consulting the line for minimum TW depth for the Type II basin, Figure 11,

$$\frac{TW}{D_1} = 11.85 \text{ for a Froude number of } 9.04$$

The tail water depth at which sweepout is incipient:

$$TW_{SO} = 11.85 \times 4.44 = 52.6 \text{ feet}$$

Adding 7 percent to this figure, the stilling basin apron should be positioned for a tail water depth of

$$52.6 + 3.7 = 56.3 \text{ feet or } 1.03D_2$$

The length of basin can be obtained by entering the intermediate curve on Figure 12 with the Froude number of 9.04.

$$\frac{L_{II}}{D_2} = 4.28$$

$$L_{II} = 4.28 \times 54.6 = 234 \text{ feet (see Figure 14)}$$

The height, width, and spacing of the chute blocks as recommended is D_1, thus the dimension can be 4 feet 6 inches.

The height of the dentated sill is $0.2D_2$ or 11 feet, while the width and spacing of the dentates can be $0.15D_2$ or 8 feet 3 inches.

SECTION 3

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

INTRODUCTION

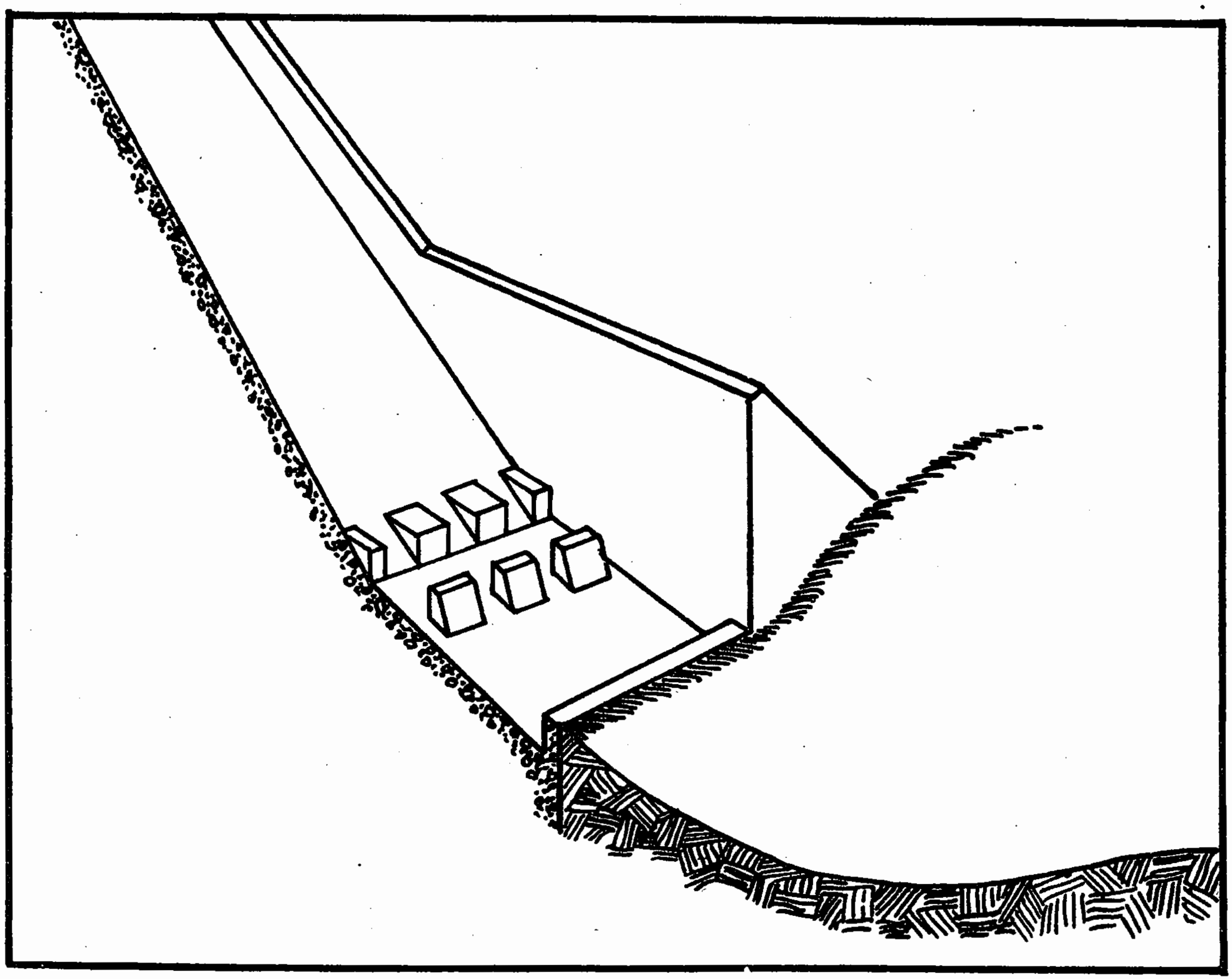
Basin II often is considered too conservative and consequently overcostly for structures carrying small discharges at moderate velocities. This can be especially true in the case of canal chutes, drops, wasteways, and other structures which are constructed by the dozen on canal systems. Any saving that can be effected in decreasing the size of these structures can amount to a sizable sum when multiplied by the number of structures involved. There is, of course, another consideration which should be kept in mind. If the dimensions of a particular structure are reduced to the point where it no longer operates satisfactorily, this mistake will be repeated many times over. 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 overall 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 [8, 16] 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.

FIGURE 16



HYDRAULIC JUMP STUDIES

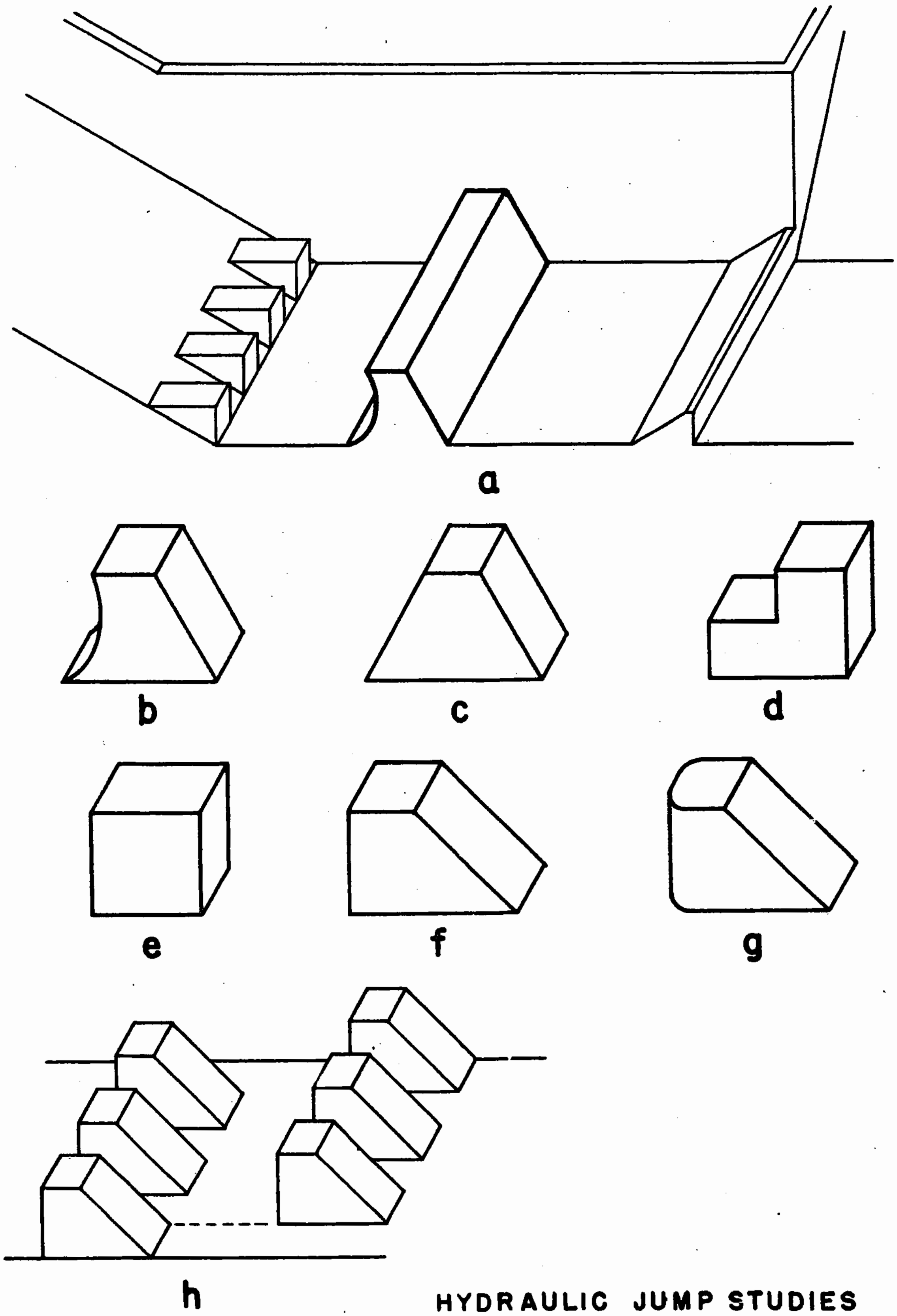
SAF STILLING BASIN

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 a 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 are 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.

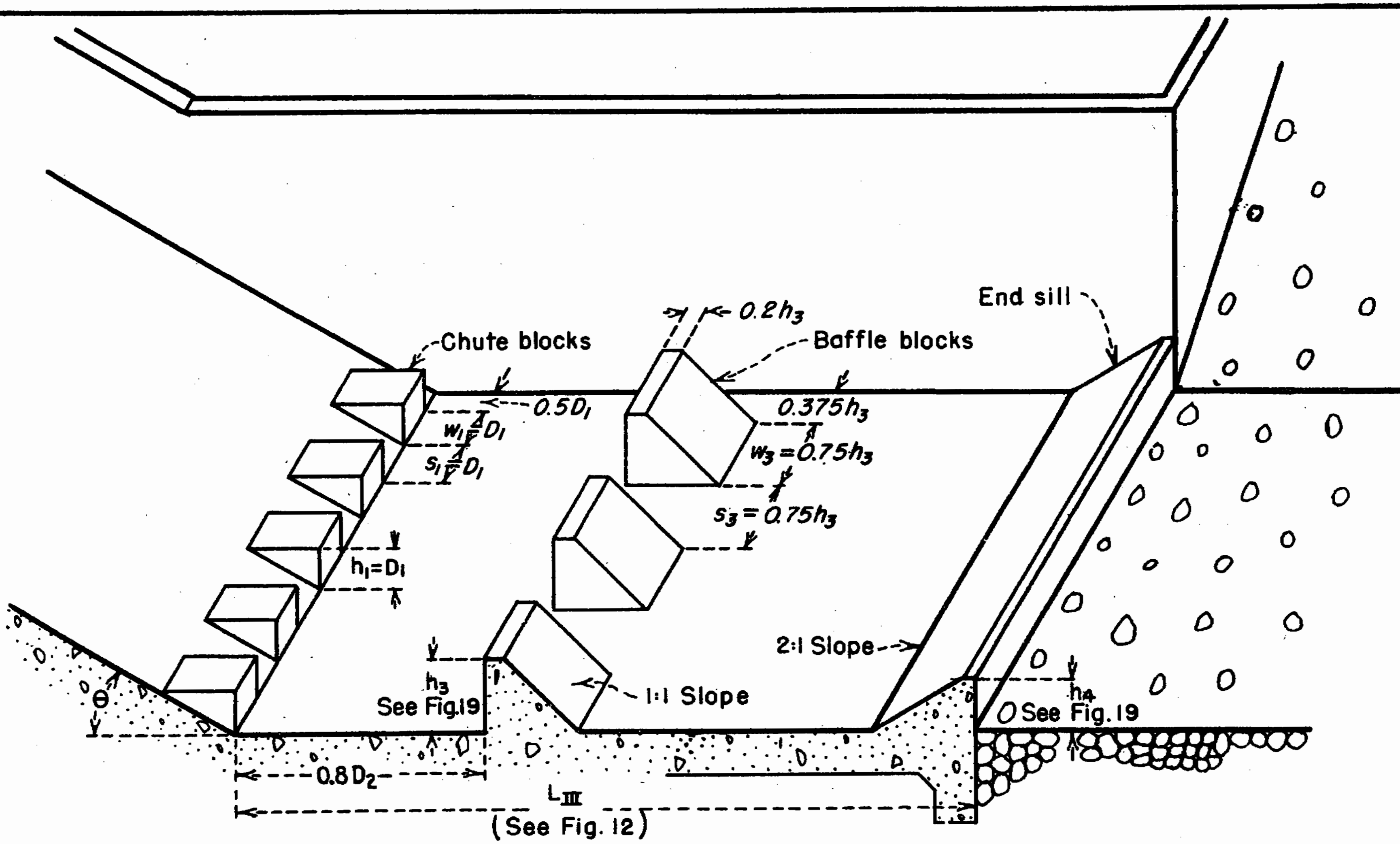
FIGURE 17



HYDRAULIC JUMP STUDIES

RECORD OF APPURTENANCES

BASIN III



HYDRAULIC JUMP STUDIES
STILLING BASIN III
RECOMMENDED PROPORTIONS

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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 tests were made on a total of 14 basins constructed in Flumes B, C, D, and E. The conditions under which the tests were run, the dimensions of the basin, and the results are recorded on Table 4. The headings are identical with those of Table 3 except for the dimensions of the baffle blocks and end sills. The additional 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 4.0. 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.

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, the same as was recommended for Basin II. 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.

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

Baffle Blocks

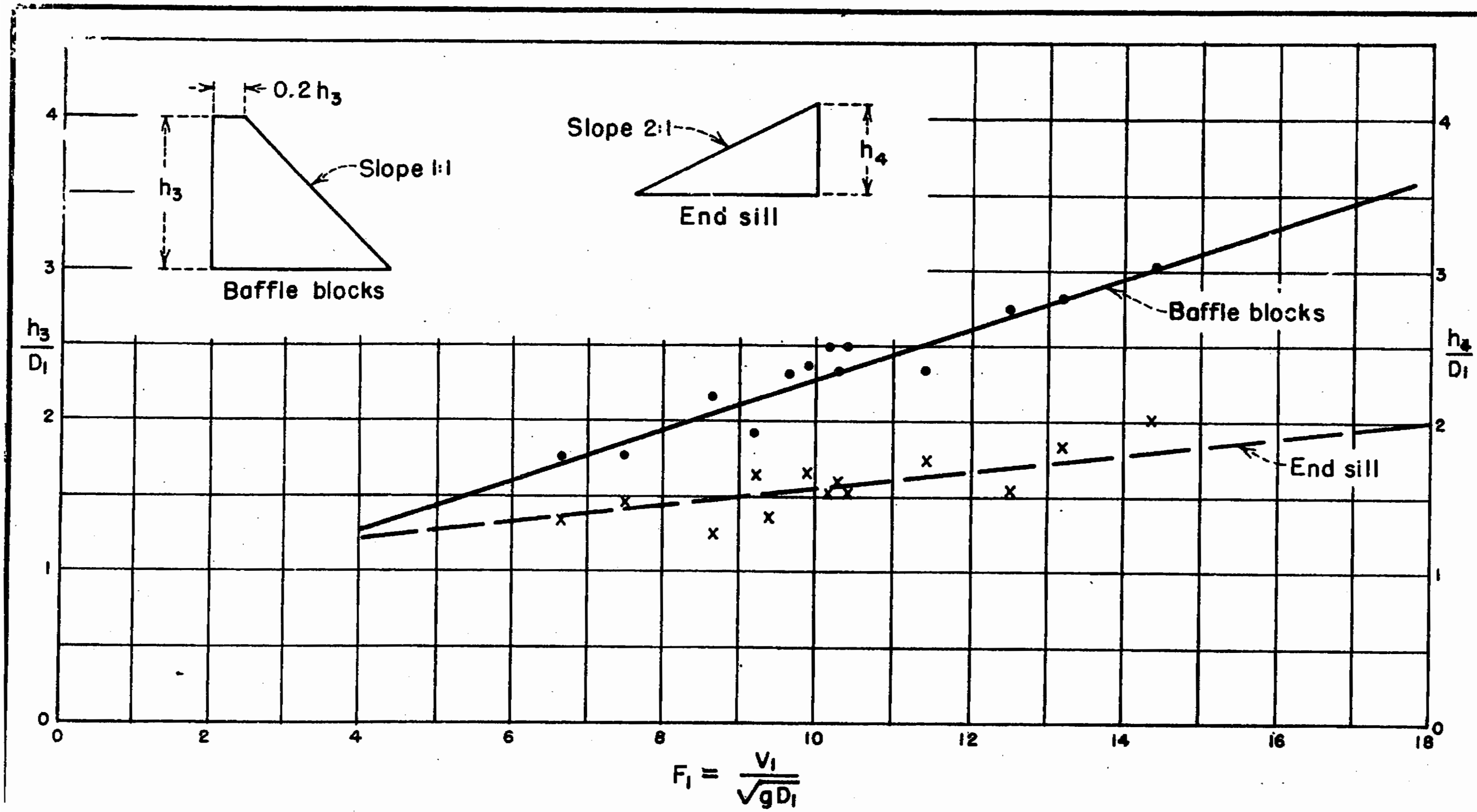
The height of the baffle blocks increases with the Froude number as can be observed from Columns 22 and 10, Table 4. The height, in terms of D_1, can be obtained from the upper line on Figure 19. The width and spacing can vary so long as the total spacing is equal to the total width of blocks. The most 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.



HYDRAULIC JUMP STUDIES

STILLING BASIN III

HEIGHT OF BAFFLE BLOCKS AND END SILL

Tail Water Depth

The SAF rules suggest the use of a tail water depth less than full conjugate depth, D_2. As in the case of Basin II, full conjugate depth, measured above the apron, is also 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 impaired, 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 advisable 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, page 37. 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, as was true of Basin II, 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

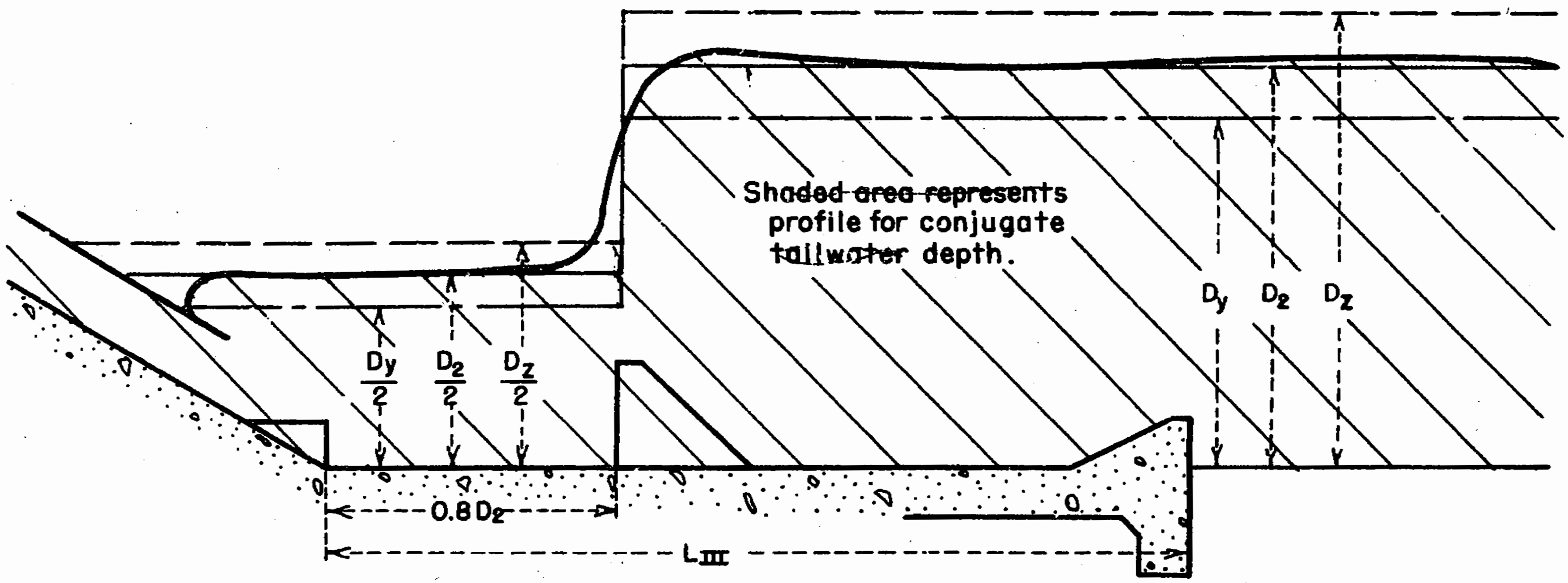
The following rules pertain to the design of the Type III basin, Figure 18:

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 (consult Section 4 for values under 4.5).

FIGURE 20



HYDRAULIC JUMP STUDIES

STILLING BASIN III

APPROXIMATE WATER SURFACE AND PRESSURE PROFILES

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 permissible.

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.

Application of Results (Example 3)

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.)

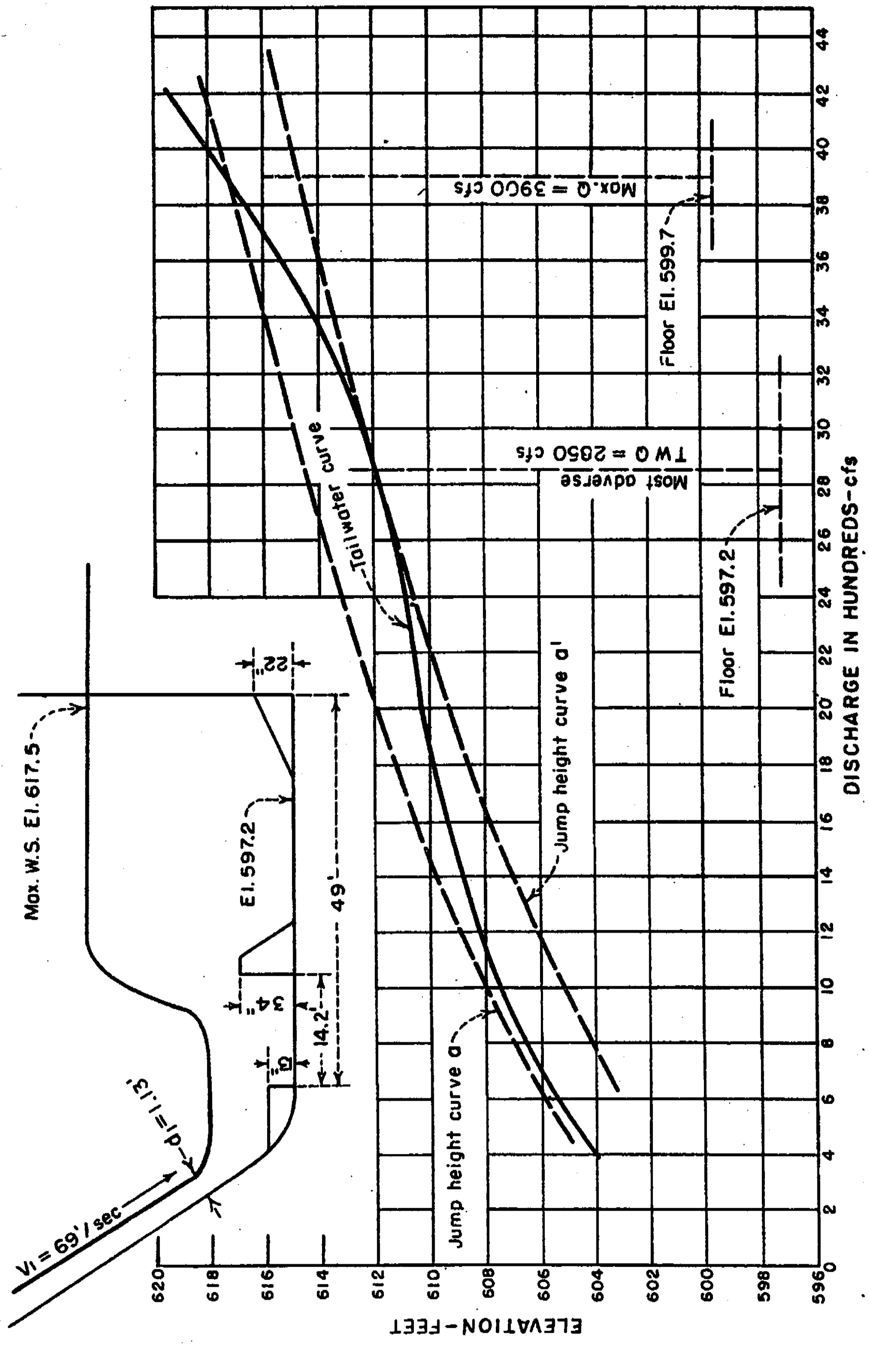
The first step is to compute the jump height curve. As V_1 and D_1 are given, the Froude number is computed and tabulated in Column 2, Table 5, below:

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	24.5	0.260	6.37	606.1	603.6

Entering Figure 11 (page 35) 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.



HYDRAULIC JUMP STUDIES
STILLING BASIN III - EXAMPLE 3
TAILWATER AND JUMP HEIGHT CURVE

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 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, page 37, 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 13 or 14 inches).

The height of the baffle blocks for a Froude number of 11.42 (Figure 19, page 53) 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 (page 49), 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, page 53) is $1.60D_1$, or

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

The final dimensions of the basin are shown on Figure 21, page 59.

SECTION 4

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

INTRODUCTION

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 NUMBER 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, page 22, 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 NUMBER 2.5 TO 4.5

Development Tests

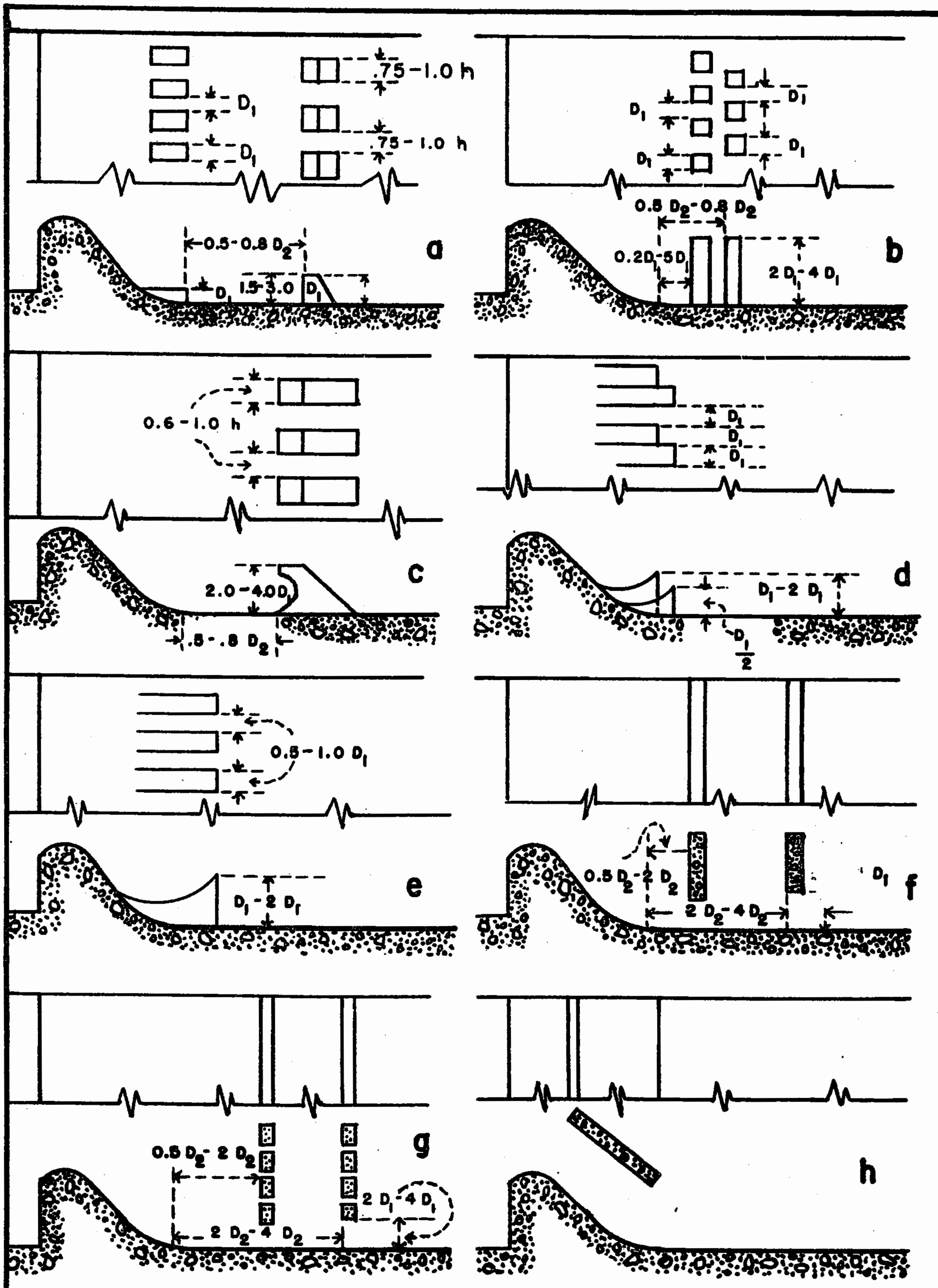
The best way to combat a wave problem is to eliminate the wave at its source; in other words, concentrate on altering the condition which generates the wave. In the case of the stilling basin preceded by an overfall or chute, two schemes were apparent for eliminating waves at their source. The first 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.

FIGURE 22



HYDRAULIC JUMP STUDIES

RECORD OF APPURTENANCES

BASIN IV.

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° 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:

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

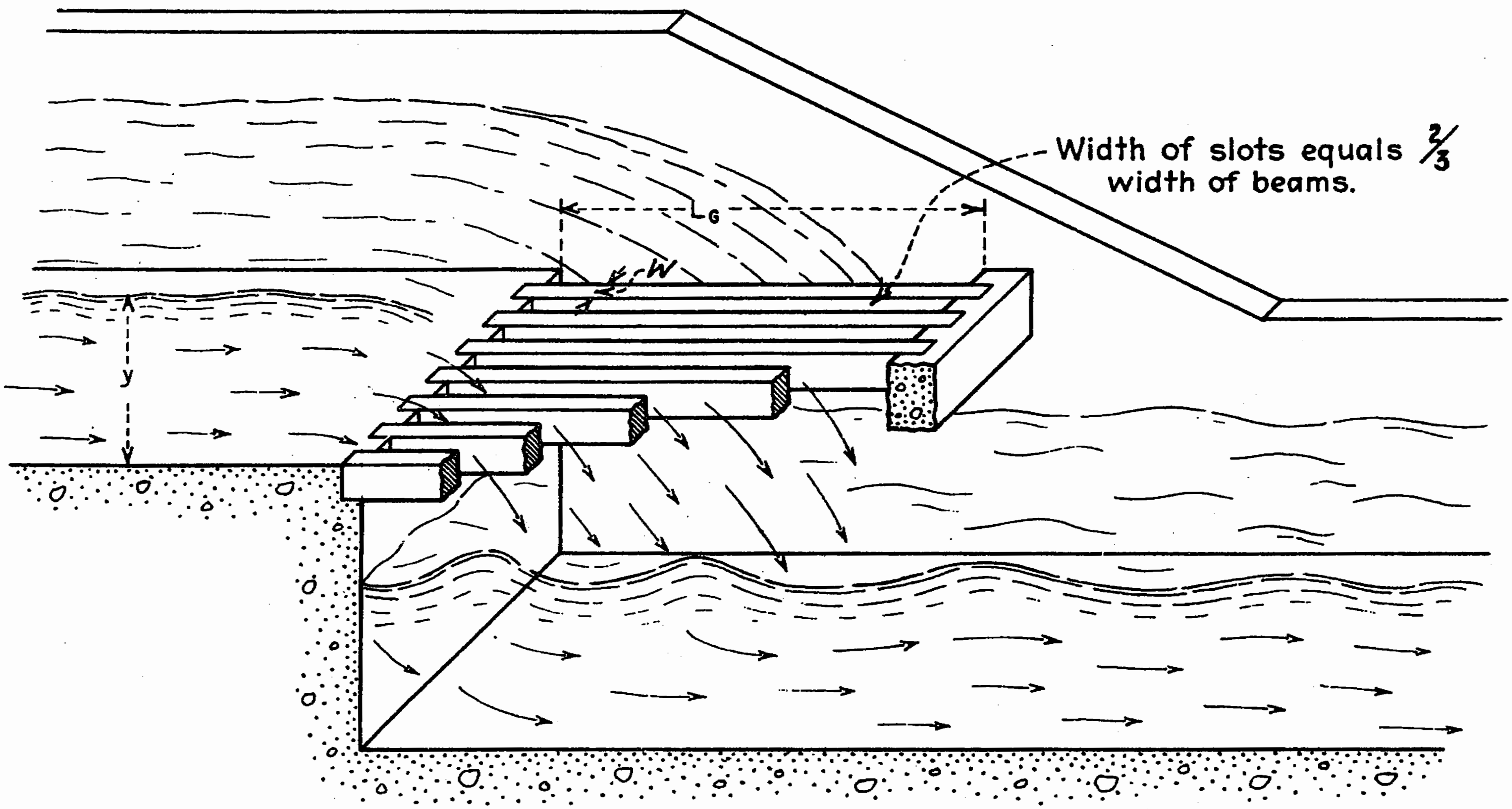
where Q is total discharge, C is an experimental coefficient, W is the width of spacing in feet, N is the number of spaces, g is the acceleration of gravity and y is the depth of flow in the canal upstream (see Figure 24). The value of C for the two arrangements tested was 0.245.

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 in the first part of Section 4 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,

FIGURE 24



HYDRAULIC JUMP STUDIES

DROP ENERGY DISSIPATOR (TYPE IV)

FOR F 2.5–4.5

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

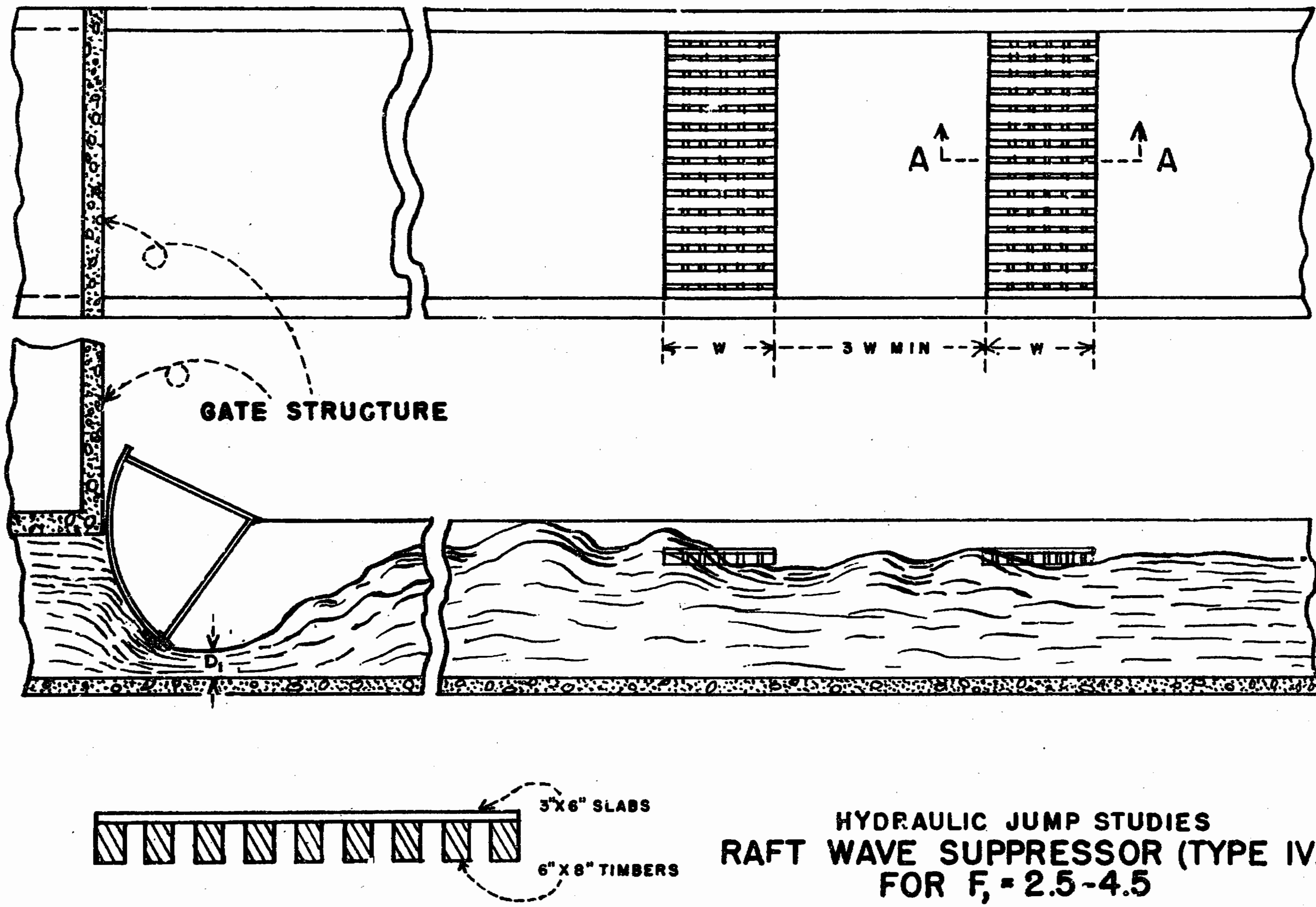
In a structure of the type shown in Figure 25, there are no means for eliminating waves at their source. Tests showed that appurtenances in the stilling basin merely produced severe splashing and created a backwater effect, resulting in submerged flow at the gate for the larger flows. Submerged flow 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.

FIGURE 25



HYDRAULIC JUMP STUDIES
RAFT WAVE SUPPRESSOR (TYPE IV.)
FOR F_1 = 2.5-4.5

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

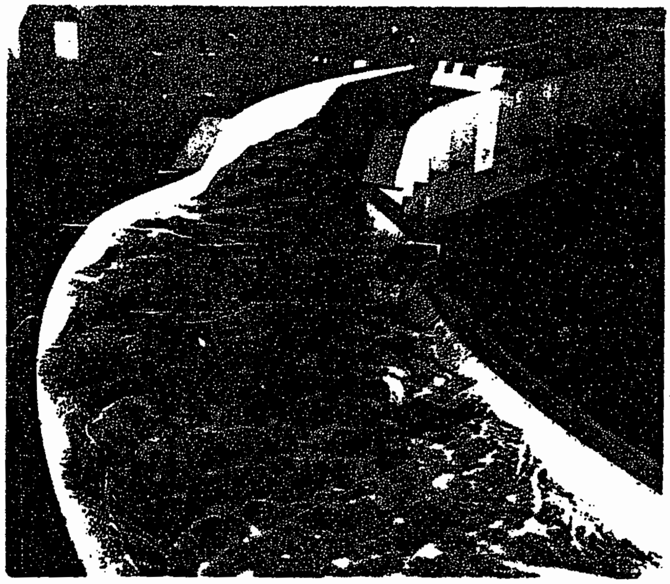
General description. By far the most effective wave dissipator is the short-tube type of underpass suppressor. The name "short-tube" is used because the structure has many of the characteristics of the short-tube discussed in hydraulics textbooks. This wave suppressor may be added to an existing structure 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.

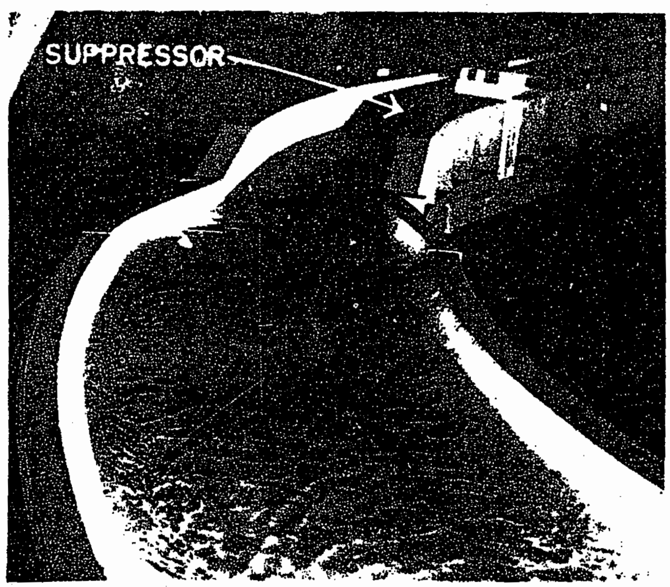
Essentially, the structure consists of a horizontal roof placed in the flow channel with a headwall sufficiently high to cause all flow to pass beneath the roof. The height of the roof above the channel floor may be set to effectively reduce wave heights for a considerable range of flows or channel stages.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

FIGURE 26



Without suppressor - waves overtop canal.



Suppressor in plac - Length 1.3D_2, submerged 30 percent

Performance of Underpass Wave Suppressor
1:32 Scale Model
Discharge 5,000 Second-feet

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.

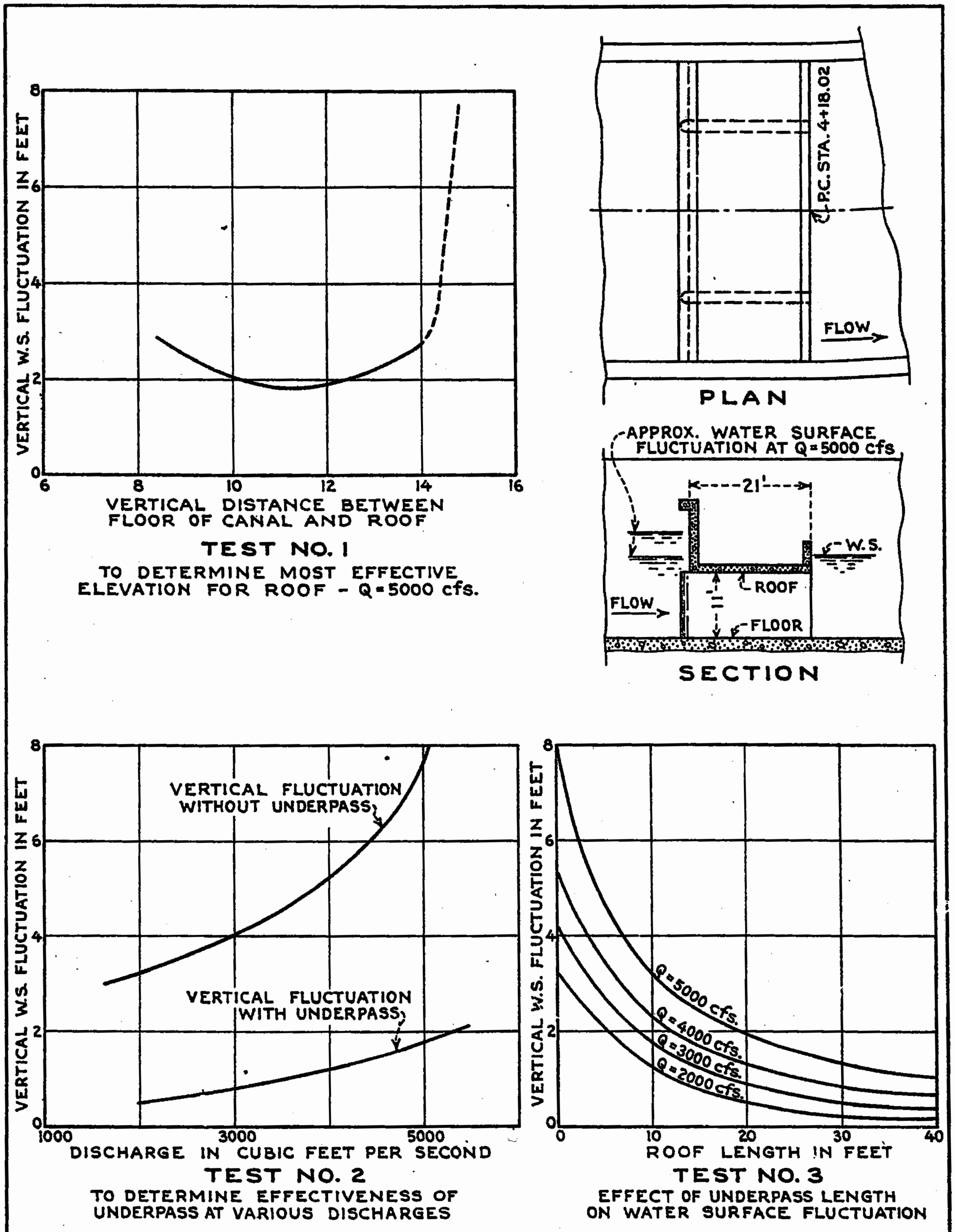
Figure 27, Test 1, shows the results of tests to determine the optimum opening between the roof and the channel floor using the maximum discharge, 5,000 second-feet. With a 14-foot opening, waves were reduced from about 8 feet to about 3 feet. Waves were reduced to less than 2 feet with an opening of 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.

FIGURE 27



WAVE SUPPRESSOR FOR
FRIANT - KERN CANAL
RESULTS OF HYDRAULIC MODEL TESTS

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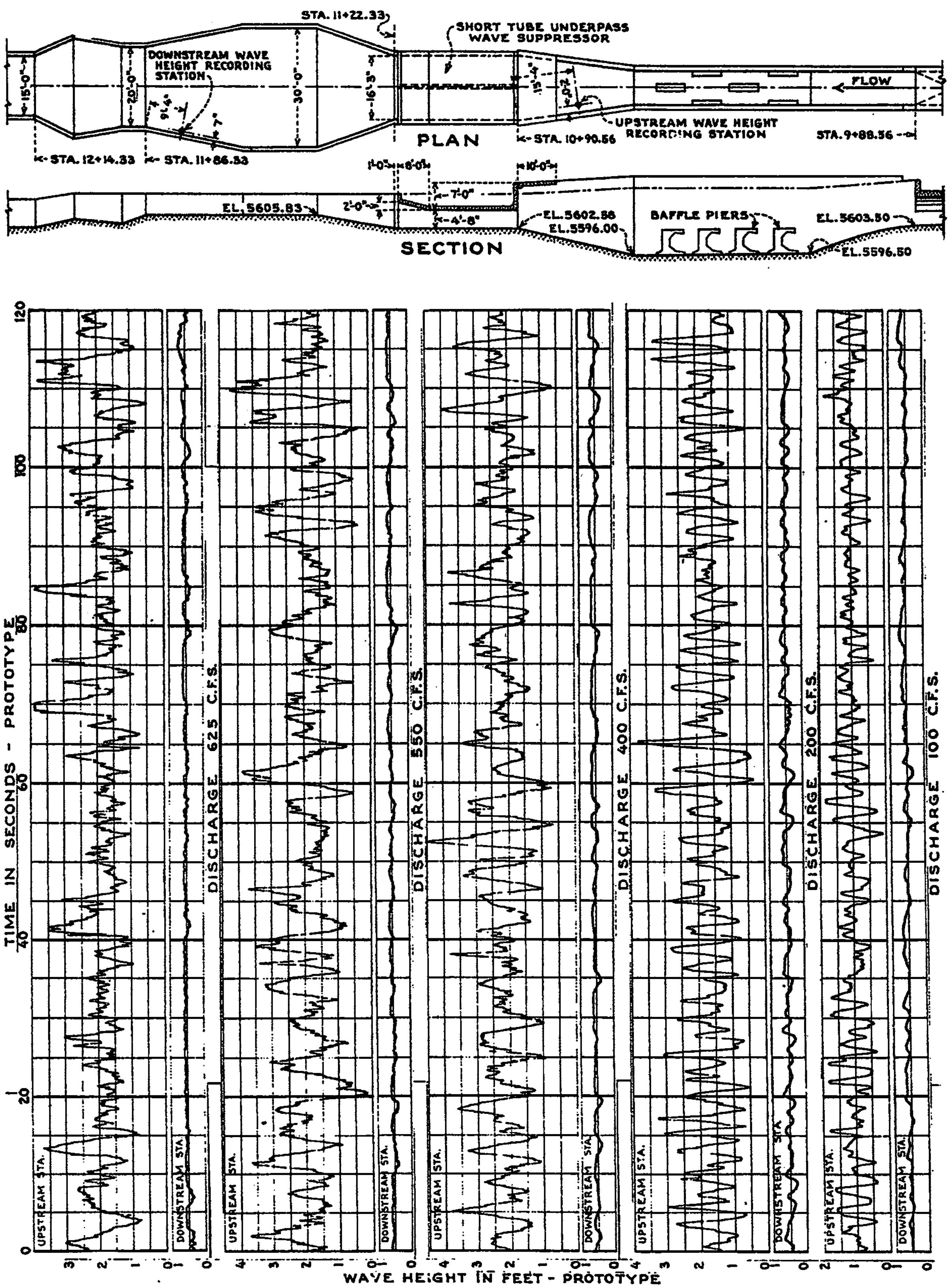


CARTER LAKE DAM NO.1 • OUTLET WORKS

WAVE HEIGHT RECORDS

(MODEL SCALE 1:16)

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.

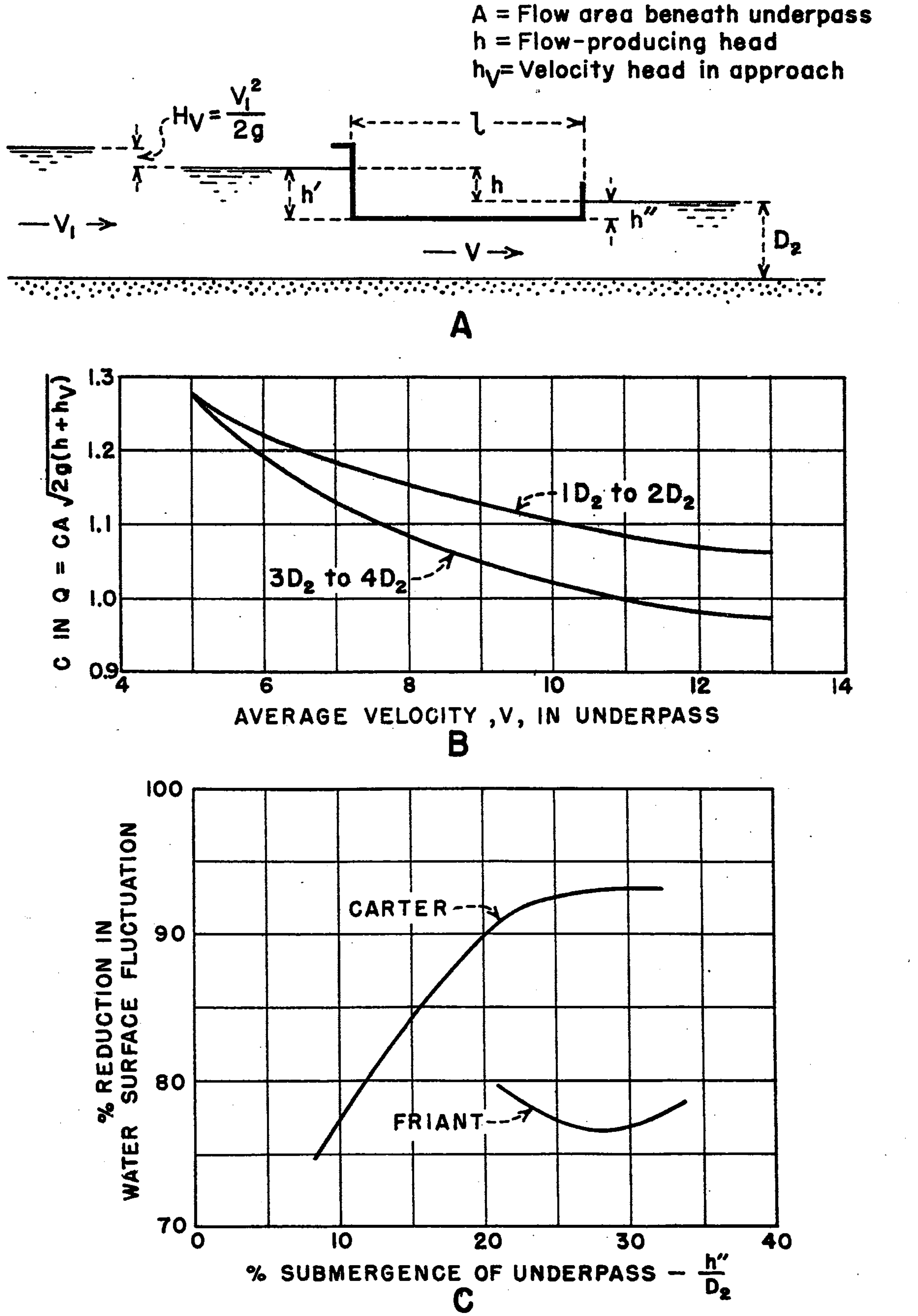
**Recorder pen reached limit of travel in this test only.

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 occurring 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



HYDRAULIC CHARACTERISTICS UNDERPASS WAVE SUPPRESSOR

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.

Sample problem, Example 4. 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 height 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

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

$h_v = 0.72$ 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 overall 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.

SECTION 5

STILLING BASIN WITH SLOPING APRON
(BASIN V)

INTRODUCTION

Much has been argued, pro and con, concerning the advantages and disadvantages of stilling basins with sloping aprons. The discussion continued indefinitely simply because there was not sufficient supporting data available from which to draw conclusions. It was decided in this study, therefore, to investigate the sloping apron basin sufficiently to answer the many debatable questions and also to provide more definite design data.

Four flumes, A, B, D, and F, Figures 1, 2, and 3, were used to obtain the range of Froude numbers desired for the tests. In the case of Flumes A, B, and D, floors were installed to the slope desired, while Flume F could be tilted to obtain slopes from 0° to 12°. The slope, as referred to in this discussion, is the tangent of the angle between the floor and the horizontal, and will be designated as "ϕ." Five principal measurements were made in these tests, namely: the discharge, the average depth of flow entering the jump, the length of the jump, the tail water depth, and the slope of the apron. The tail water was adjusted so that the front of the jump formed either at the intersection of the spillway face and the sloping apron or, in the case of the tilting flume, at a selected point.

The jump that occurs on the sloping apron takes many forms depending on the slope and arrangement of the apron, the value of the Froude number, and the concentration of flow or discharge per foot of width; but from all appearances, the dissipation is as effective as occurs in the true hydraulic jump on a horizontal apron.

PREVIOUS EXPERIMENTAL WORK

Previous experimental work on the sloping apron has been carried on by several experimenters. In 1934, the late C. L. Yarnell of the United States Department of Agriculture supervised a series of experiments on the hydraulic jump on sloping aprons. Carl Kindsvater[5] later compiled these data and presented a rather complete picture, both experimentally and theoretically, for one slope, namely: 1:6 ($\tan \phi = 0.167$). G. H. Hickox[5] presented data for a series of experiments on a slope of 1:3 ($\tan \phi = 0.333$). Bakhmeteff[1] and Matzke[6] performed experiments on slopes of 0 to 0.07 in a flume 6 inches wide.

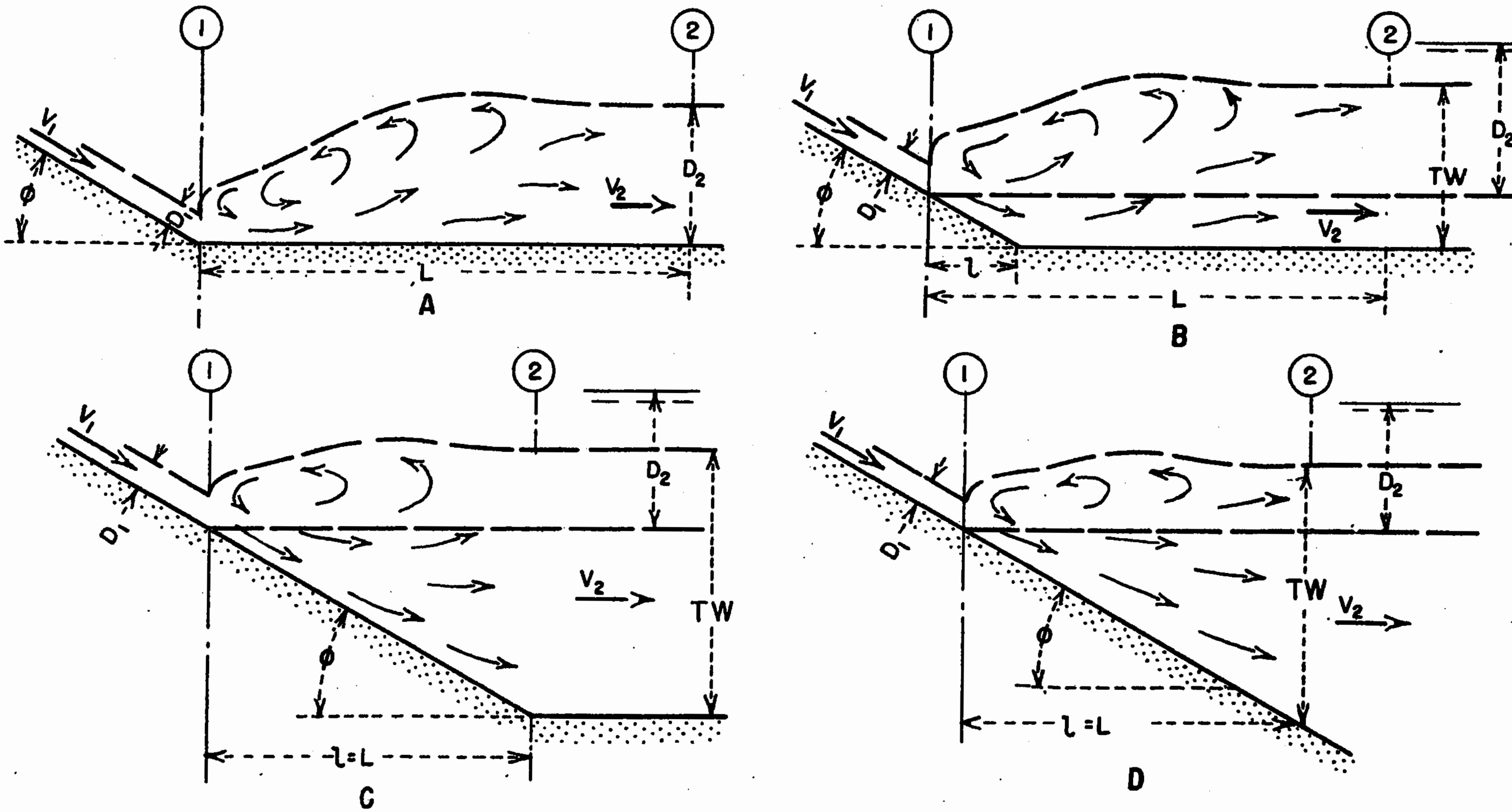
From an academic standpoint, the jump may occur in several ways on a sloping apron, as outlined by Kindsvater, presenting separate and distinct problems, Figure 30. Case A is a jump on a horizontal apron. In Case B, the toe of the jump forms on the slope, while the end occurs over the horizontal apron. In Case C, the toe of the jump is on the slope, and the end is at the junction of the slope and the horizontal apron; while in Case D, the entire jump forms on the slope. With so many possibilities, it is easily understood why experimental data have been lacking on the sloping apron. Messrs. Yarnell, Kindsvater, Bakhmeteff, and Matzke limited their experiments to Case D. B. D. Rindlaub[7] of the University of California concentrated on the solution of Case B, but his experimental results are complete for only one slope, that of 12.33° ($\tan \phi = 0.217$).

SLOPING APRON TESTS

From a practical standpoint, the scope of the present test program need not be so broad as outlined in Figure 30. For example, the action in Cases C and D is for all practical purposes the same, if it is assumed that a horizontal floor begins at the end of the jump for Case D. The first of the current experiments to be described in this chapter involves Case D. However, sufficient tests were made on Case C to verify the above statement that Cases C and D can be considered as one. The second set of tests will deal with Case B. Case B is virtually Case A operating with excessive tail water depth. As the tail water depth is further increased, Case B approaches Case C. The results of Case A have already been discussed in the preceding chapters, and Cases D and B will be considered here in order.

Tail Water Depth (Case D)

Data obtained from the four flumes used in the sloping apron tests (Case D experiments) are tabulated in Table 8. The headings are very much the same as those in previous tables, but will need some explanation. Column 2 lists the tangents of the angles of the slopes tested. The depth of flow entering the jump, D_1, Column 8, was measured at the beginning of the jump in each case, corresponding to Section 1, Figure 30. It represents the average of a generous number of point gage measurements. The velocity at this same point, V_1, Column 7, was computed by dividing the unit discharge, q (Column 5), by D_1. The length of jump, Column 11, was measured in the flume, bearing in mind that the object of the test was to obtain practical data for stilling basin design. The end of the jump was chosen as the point where the high velocity jet began to lift from the floor, or a point on the level tail water surface immediately downstream from the surface roller, whichever



HYDRAULIC JUMP STUDIES
STILLING BASIN V
SLOPING APRONS

FIGURE 30

Table 8

STILLING BASINS WITH SLOPING APRON
Case D, Basin V

Test flume (1)	Slope of apron tan ϕ (2)	Q Total discharge cfs (3)	W Width of basin ft (4)	q Discharge per foot of basin cfs (5)	TW Tail-water depth ft (6)	V_1 Velocity entering jump ft/sec (7)	D_1 Depth entering jump ft (8)	$\frac{TW}{D_1}$ (9)	$F_1 = \frac{V_1}{\sqrt{gD_1}}$ (10)	L Length of jump ft (11)	$\frac{L}{TW}$ (12)	$\frac{D_2}{D_1}$ (13)	D_2 Conjugate TW depth ft (14)	$\frac{TW}{D_2}$ (15)	$\frac{L}{D_2}$ (16)	K Shape factor (17)
A	0.067	2.000	4.880	0.410	0.520	7.88	0.052	10.00	6.09	2.60	5.00	8.20	0.426	1.22	6.11	2.50
		2.250		0.461	0.560	8.09	0.057	9.82	5.97	2.90	5.18	7.90	0.450	1.24	6.45	2.50
		2.500		0.512	0.589	8.26	0.062	9.50	5.85	3.10	5.26	7.85	0.486	1.21	6.38	2.40
		2.750		0.564	0.629	8.42	0.067	9.39	5.73	3.30	5.25	7.70	0.516	1.22	5.40	2.45
		3.000		0.615	0.660	8.54	0.072	9.17	5.61	3.40	5.15	7.55	0.544	1.21	6.25	2.45
		3.250		0.666	0.694	8.65	0.077	9.01	5.49	3.45	4.97	7.40	0.570	1.22	5.05	2.50
		3.500		0.717	0.744	8.74	0.082	9.07	5.38	3.60	4.84	7.20	0.590	1.26	6.10	2.80
		1.500	4.350	0.345	0.474	7.67	0.045	10.53	6.37	2.40	5.06	8.60	0.387	1.22	6.20	2.50
		2.500		0.575	0.642	8.46	0.068	9.44	5.72	3.20	4.98	7.70	0.523	1.23	6.12	2.50
		3.500		0.805	0.792	8.85	0.091	8.70	5.17	4.00	5.05	6.90	0.628	1.26	6.37	2.75
	0.096	2.000	4.830	0.414	0.560	7.96	0.052	10.77	6.15	2.50	4.47	8.20	0.426	1.31	5.87	2.04
		2.500		0.518	0.652	7.97	0.065	10.03	5.51	3.60	5.52	7.45	0.484	1.35	7.44	2.28
		3.000		0.621	0.745	8.28	0.075	9.93	5.33	3.20	4.30	7.10	0.532	1.40	6.01	2.40
		3.500		0.725	0.835	8.53	0.085	9.82	5.15	3.60	4.31	6.90	0.586	1.42	6.15	2.50
		4.000		0.828	0.940	8.63	0.096	9.79	4.90	4.00	4.26	6.50	0.624	1.51	6.41	2.75
	0.135	2.000	4.810	0.416	0.620	6.93	0.060	10.33	4.99	2.50	4.06	6.60	0.396	1.56	6.32	2.15
		2.500		0.520	0.710	7.54	0.069	10.29	5.06	3.00	4.23	6.75	0.466	1.52	6.44	2.07
		3.000		0.624	0.895	7.80	0.080	10.06	4.86	3.20	3.97	6.40	0.512	1.57	6.25	2.15
		3.500		0.728	0.905	8.09	0.090	10.06	4.75	3.60	3.98	6.30	0.567	1.60	6.34	2.22
		4.000		0.832	0.985	8.58	0.097	10.15	4.85	3.90	3.96	6.40	0.621	1.59	6.28	2.19
	0.152	1.500	4.350	0.345	0.540	6.27	0.055	9.82	4.71	2.10	3.89	6.20	0.341	1.58	6.16	1.94
		2.000		0.460	0.663	6.76	0.068	9.75	4.57	2.55	3.85	6.10	0.415	1.60	6.15	2.00
		2.500		0.575	0.790	7.57	0.076	10.39	4.84	3.10	3.92	6.45	0.490	1.61	6.33	2.00
		3.000		0.690	0.900	7.67	0.090	10.00	4.50	3.40	3.78	6.00	0.540	1.67	6.30	2.10
	0.185	1.500	4.350	0.345	0.600	6.05	0.057	10.53	4.47	2.15	3.58	5.90	0.336	1.78	6.40	1.83
		2.000		0.460	0.720	6.57	0.070	10.29	4.38	2.60	3.61	5.80	0.406	1.77	6.40	1.83
		2.500		0.575	0.840	7.01	0.082	10.24	4.31	3.00	3.57	5.70	0.467	1.80	6.42	1.85
	0.218	1.750	4.350	0.402	0.700	6.00	0.067	10.45	4.08	2.30	3.29	5.45	0.365	1.92	6.30	1.70
		2.250		0.517	0.862	6.63	0.078	11.05	4.19	2.70	3.13	5.55	0.433	1.99	6.24	1.73
	0.280	1.250	4.350	0.287	0.620	4.70	0.061	10.16	3.35	1.60	2.58	4.25	0.259	2.39	6.18	1.44
		1.500		0.345	0.675	4.79	0.072	9.38	3.15	1.80	2.67	4.05	0.292	2.31	6.17	1.44
		1.750		0.402	0.752	4.79	0.084	8.95	2.91	1.95	2.59	3.70	0.311	2.42	6.27	1.46
B	0.052	1.000	2.000	0.500	0.855	17.24	0.029	29.48	17.85	4.10	4.79	24.75	0.718	1.19	5.71	2.94
		1.500		0.750	1.010	16.30	0.046	21.96	13.40	5.10	5.05	18.45	0.849	1.19	6.01	2.80
		2.000		1.000	1.160	16.39	0.061	19.02	11.69	6.10	5.26	16.10	0.982	1.18	6.21	2.78
		2.500		1.250	1.300	17.12	0.073	17.81	11.16	6.50	5.00	15.35	1.121	1.16	5.80	2.45
		3.000		1.500	1.426	17.05	0.088	16.20	10.13	7.50	5.26	13.85	1.218	1.17	6.15	2.70
		3.500		1.750	1.570	17.16	0.102	15.39	9.46	8.00	5.10	12.95	1.321	1.19	6.06	2.80
		4.000		2.000	1.693	17.09	0.117	14.47	8.80	8.90	5.26	12.10	1.416	1.20	6.28	2.92
		4.500		2.250	1.813	17.05	0.132	13.73	8.27	9.60	5.29	11.30	1.492	1.22	6.44	3.10
		5.000		2.500	1.920	17.01	0.147	13.06	7.82	9.80	5.10	10.60	1.558	1.23	6.29	3.20
		5.500		2.750	2.020	17.08	0.161	12.55	7.50	10.50	5.20	10.20	1.642	1.23	6.40	3.20
		6.000		3.000	2.110	16.95	0.177	11.92	7.10	11.00	5.21	9.65	1.708	1.24	6.44	3.30
	0.102	1.000		0.500	0.970	15.63	0.032	30.31	15.40	4.20	4.33	21.25	0.680	1.42	6.17	2.51
		1.500		0.750	1.180	15.63	0.048	24.58	12.57	5.20	4.41	17.30	0.830	1.42	6.27	2.50
		2.000		1.000	1.354	15.87	0.063	21.49	11.14	6.10	4.51	15.35	0.967	1.40	6.31	2.44
		2.500		1.250	1.543	16.23	0.077	20.04	10.30	6.80	4.40	14.15	1.088	1.42	6.24	2.50
		3.000		1.500	1.724	16.48	0.091	18.95	9.63	7.60	4.41	13.20	1.200	1.44	6.34	2.56
		3.000		1.500	1.720	16.30	0.092	18.70	9.47	7.50	4.36	12.95	1.191	1.44	6.30	2.58
		3.500		1.750	1.890	16.36	0.107	17.66	8.81	8.20	4.34	12.10	1.293	1.46	6.34	2.75
		4.000		2.000	2.040	16.53	0.121	16.86	8.37	8.80	4.31	11.40	1.379	1.48	6.38	2.72
		4.500		2.250	2.152	16.42	0.137	15.71	7.82	9.40	4.37	10.60	1.452	1.48	6.47	2.70
		5.000		2.500	2.300	16.45	0.152	15.13	7.44	10.00	4.34	10.10	1.536	1.50	6.51	2.75
		5.500		2.750	2.450	16.18	0.170	14.41	6.91	10.60	4.33	9.35	1.590	1.54	6.67	2.85

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Table 8—Continued

STILLING BASINS WITH SLOPING APRON
Case D, Basin V

Test flume (1)	Slope of apron tan ϕ (2)	Q Total discharge cfs (3)	W Width of basin ft (4)	q Discharge per foot of basin cfs (5)	TW Tail-water depth ft (6)	V_1 Velocity entering jump ft/sec (7)	D_1 Depth entering jump ft (8)	$\frac{TW}{D_1}$ (9)	$F_1 = \frac{V_1}{\sqrt{gD_1}}$ (10)	L Length of jump ft (11)	$\frac{L}{TW}$ (12)	$\frac{D_2}{D_1}$ (13)	D_2 Conjugate TW depth ft (14)	$\frac{TW}{D_2}$ (15)	$\frac{L}{D_2}$ (16)	K Shape factor (17)
B	0.164	2.000	2.000	1.000	1.537	15.38	0.065	23.65	10.64	6.10	3.97	14.65	0.952	1.61	6.41	1.88
		2.500		1.250	1.737	14.88	0.084	20.68	9.05	6.90	3.97	12.40	1.042	1.67	6.62	1.95
		3.000		1.500	1.940	14.71	0.102	19.02	8.11	7.50	3.86	11.05	1.128	1.72	6.64	2.02
		3.500		1.750	2.120	14.83	0.118	17.97	7.61	8.20	3.87	10.30	1.215	1.74	6.75	2.03
		4.000		2.000	2.270	15.04	0.133	17.07	7.27	8.70	3.83	9.85	1.310	1.73	6.64	2.01
		4.500		2.250	2.420	14.90	0.151	16.03	6.75	9.20	3.80	9.10	1.374	1.76	6.70	2.08
		5.000		2.500	2.590	14.88	0.168	15.42	6.39	9.70	3.74	8.65	1.454	1.78	6.67	2.08
		5.500		2.750	2.750	14.86	0.185	14.86	6.09	10.20	3.71	8.20	1.517	1.81	6.73	2.10
	0.213	2.000		1.000	1.750	13.33	0.075	23.33	8.60	6.00	3.43	11.75	0.881	1.99	6.81	1.71
		2.500		1.250	2.000	13.59	0.092	21.74	7.89	6.60	3.30	10.70	0.984	2.03	6.71	1.76
		3.000		1.500	2.150	13.51	0.111	19.37	7.15	7.30	3.40	9.70	1.077	2.00	6.78	1.73
		3.500		1.750	2.370	13.57	0.129	18.37	6.65	8.00	3.38	9.00	1.161	2.04	6.89	1.76
		4.000		2.000	2.600	13.51	0.148	17.57	6.19	8.30	3.19	8.35	1.236	2.10	6.71	1.79
		4.500		2.250	2.720	13.55	0.166	16.39	5.86	9.10	3.34	7.85	1.303	2.09	6.98	1.78
		5.000		2.500	2.890	13.59	0.184	15.71	5.58	9.60	3.32	7.50	1.380	2.09	6.96	1.79
		5.500		2.750	3.100	13.55	0.203	15.27	5.30	10.00	3.22	7.10	1.441	2.15	6.94	1.81
	0.263	2.000		1.000	1.900	11.63	0.086	22.09	6.98	5.60	2.95	9.45	0.813	2.34	6.89	1.55
		3.000		1.500	2.330	11.63	0.129	18.06	5.70	6.90	2.96	7.65	0.987	2.36	6.99	1.56
		4.000		2.000	2.820	12.35	0.162	17.41	5.40	8.10	2.87	7.25	1.174	2.40	6.90	1.57
		5.000		2.500	3.270	12.38	0.202	16.19	4.85	9.20	2.81	6.45	1.303	2.51	7.06	1.59
		6.000		3.000	3.602	12.35	0.243	14.82	4.41	10.00	2.77	5.80	1.409	2.56	7.09	1.59
D	0.100	4.000	3.970	1.007	1.530	18.64	0.054	28.33	14.14	6.60	4.31	19.50	1.053	1.45	6.27	2.65
		6.000		1.511	1.888	19.12	0.079	23.90	11.99	8.20	4.34	16.50	1.303	1.45	6.29	2.65
		8.000		2.015	2.200	19.75	0.102	21.57	10.90	9.70	4.41	14.95	1.525	1.44	6.36	2.65
		10.000		2.518	2.630	20.14	0.125	21.04	10.04	11.50	4.37	13.75	1.719	1.53	6.69	2.85
		[illegible]250		0.567	1.200	18.90	0.030	40.00	19.23	4.75	3.96	26.70	0.801	1.50	5.93	2.75
		[illegible].500		1.134	1.710	18.29	0.062	27.58	12.94	7.80	4.56	17.90	1.109	1.54	7.03	2.90
		6.750		1.700	2.100	19.54	0.087	24.14	11.67	9.10	4.33	16.10	1.400	1.50	6.50	2.78
F	0.174	1.980	1.000	1.980	1.452	7.17	0.276	5.26	2.41	4.30	2.96	3.00	0.828	1.75	5.19	1.88
		2.800		2.800	1.663	7.69	0.364	4.57	2.24	5.00	3.01	2.80	1.018	1.63	4.91	1.76
	0.200	2.980		2.980	2.035	8.32	0.358	5.68	2.45	5.80	2.85	3.05	1.092	1.86	5.31	1.72
		3.850		3.850	2.460	8.48	0.454	5.42	2.22	6.70	2.72	2.75	1.248	1.97	5.37	1.81
	0.150	3.850		3.850	2.095	7.97	0.483	4.33	2.02	5.90	2.82	2.45	1.183	1.77	4.99	2.10
		1.780		1.780	1.260	6.93	0.257	4.90	2.41	4.00	3.17	3.00	0.771	1.63	5.19	2.00
	0.100	1.940		1.940	1.180	6.40	0.303	3.89	2.05	3.70	3.14	2.50	0.757	1.56	4.89	2.93
		3.870		3.870	1.648	7.38	0.524	3.14	1.80	4.80	2.91	2.15	1.126	1.46	4.26	2.55
	0.050	3.620		3.620	1.357	7.62	0.475	2.86	1.95	4.30	3.17	2.35	1.116	1.22	3.85	3.00
		1.820		1.820	1.306	12.38	0.147	8.88	5.69	6.80	5.21	7.65	1.124	1.16	6.05	3.90
		3.910		3.910	1.291	6.66	0.587	2.20	1.53	3.60	2.79	1.80	1.057	1.22	3.41	3.20
		2.300		2.300	0.943	5.87	0.392	2.41	1.65	2.80	2.97	1.95	0.764	1.23	3.67	3.20

occurred farthest downstream. The length of the jump, as tabulated in Column 11, is the horizontal distance from Sections 1 to 2, Figure 30. The tail water depth, tabulated in Column 6, is the depth measured at the end of the jump, corresponding to the depth at Section 2 on Figure 30.

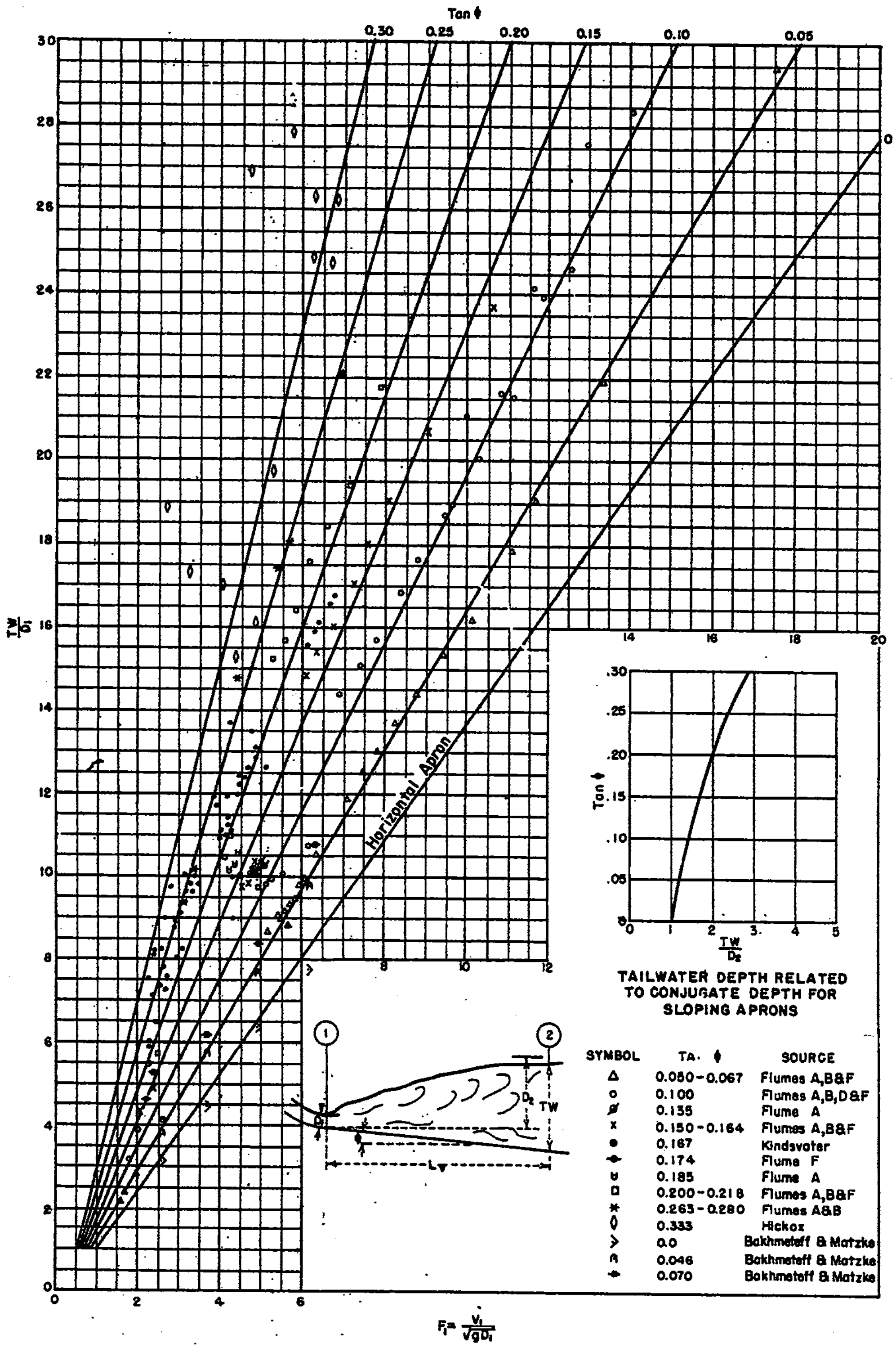
The ratio $\frac{TW}{D_1}$ (Column 9, Table 8) is plotted with respect to the Froude number (Column 10) for sloping aprons having tangents 0.05 to 0.30 on Figure 31. The plot for the horizontal apron ($\tan \phi = 0$) is the same as shown in Figure 5. Superimposed on Figure 31 are data from Kindsvater,[5] Hickox,[5] Bahkmeteff,[1] and Matzke.[6] The agreement is within experimental error.

The small chart on Figure 31 was constructed using data from the larger chart, and shows, for a range of apron slopes, the ratio of tail water depth for a continuous sloping apron, to conjugate depth for a horizontal apron. As indicated on the small sketch in Figure 31, D_2 and TW are identical for a horizontal apron. The conjugate depth, D_2 listed in Column 14, Table 6, is the depth necessary for a jump to form on an imaginary horizontal floor beginning at Section 1, Figure 31.

The small chart, therefore, shows the extra depth, required for a jump of a given Froude number to form on a sloping apron, rather than on a horizontal apron. For example, if the tangent of the slope is 0.10, a tail water depth equal to 1.4 times the conjugate depth (D_2 for a horizontal apron) will occur at the end of the jump; while if the slope is 0.30, the tail water depth at the end of the jump will be 2.8 times the conjugate depth D_2. The conjugate depth D_2 used in connection with a sloping apron is merely a convenient reference figure which has no other meaning. It will be used throughout this discussion on sloping aprons.

Length of Jump (Case D)

The length of jump for the Case D experiments has been presented in two ways. First, the ratio length of jump to tail water depth, Column 12, was plotted with respect to the Froude number on Figure 32 for sloping aprons having tangents from 0 to 0.25. Secondly, the ratio of length of jump to the conjugate tail water depth, Column 16, Table 8, has been plotted with respect to the Froude number for the same range of slopes on Figure 33. Although not evident on Figure 32, it can be seen from Figure 33 that the length of jump on a sloping apron is longer than that which occurs on a horizontal floor. For example, for a Froude number of 8, the ratio $\frac{L}{D_2}$ varies from 6.1, for a horizontal apron, to 7.0, for an apron with a slope of 0.25. Length determinations from Kindsvater[5] for a slope of 0.167 are also plotted on Figure 32. The points show a wide spread.



HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE D)

HYDRAULIC JUMP ON SLOPING APRON

RATIO OF TAILWATER DEPTH TO D_1

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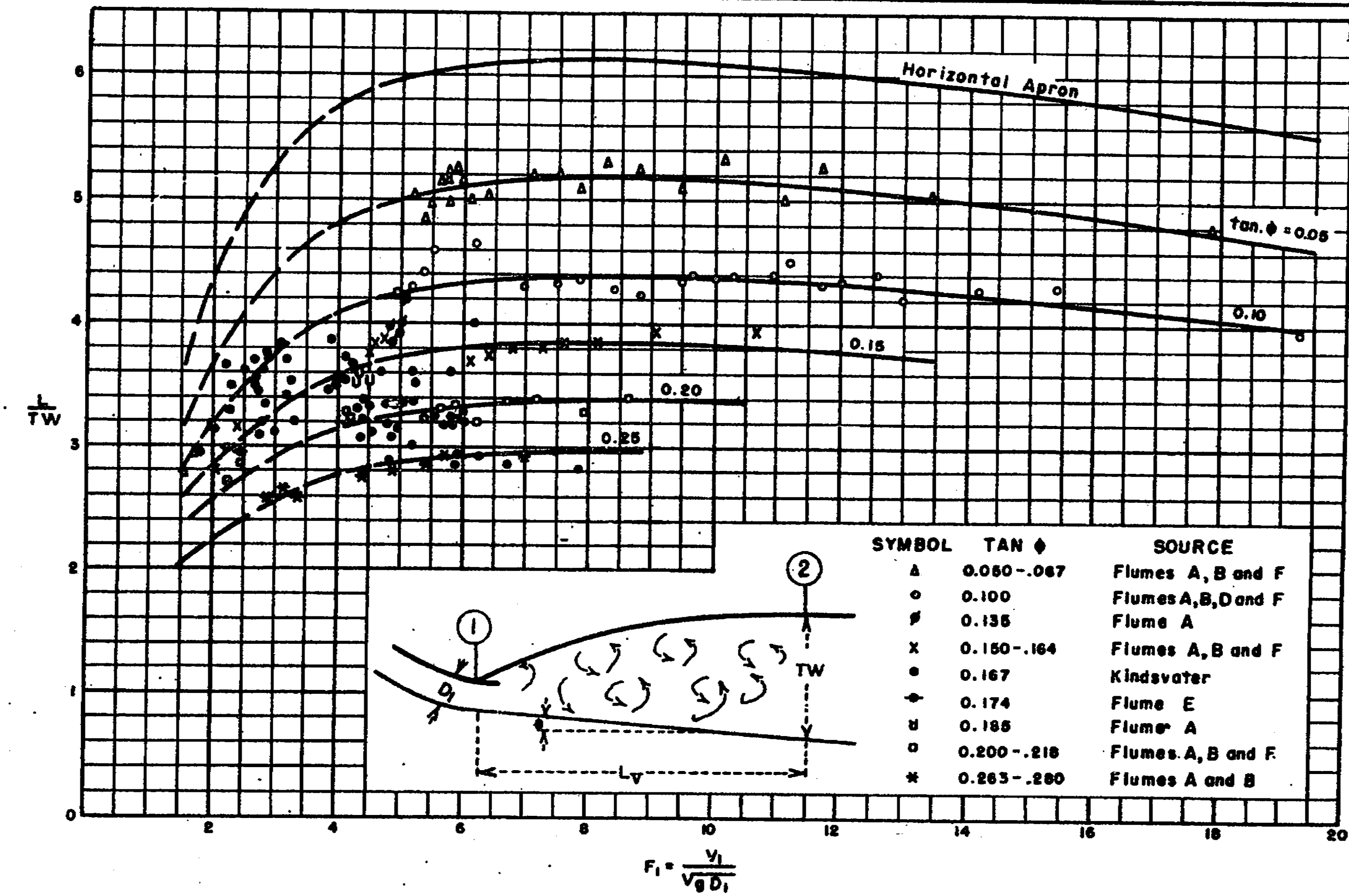
FIGURE 32



HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE D)

LENGTH OF JUMP IN TERMS OF TW DEPTH

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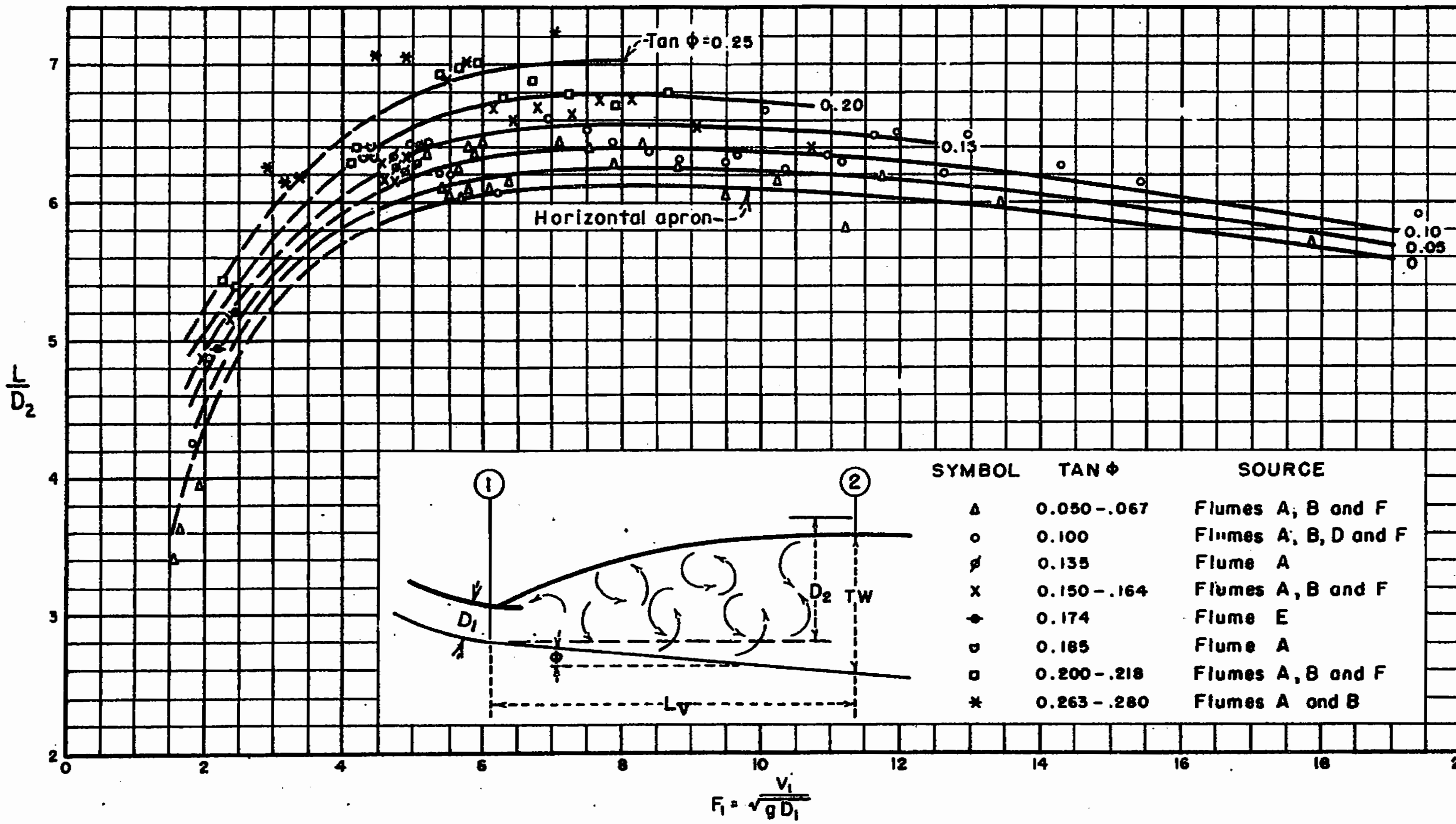


HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE D)

LENGTH OF JUMP IN TERMS OF CONJUGATE DEPTH, D_2

FIGURE 33

Expression for Jump on Sloping Apron (Case D)

Several mathematicians and experimenters have developed expressions for the hydraulic jump on sloping aprons,[2 5 6 13] so there is no need to repeat any of these derivations here. An expression presented by Kindsvater[5] is the more common and perhaps the more practical to use.

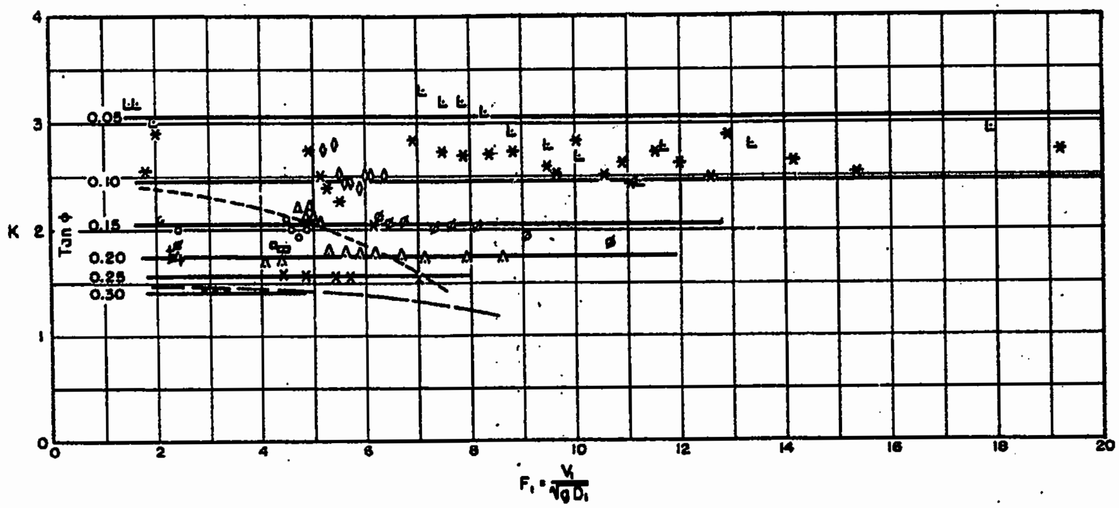
$$\frac{D_2}{D_1} = \frac{1}{2 \cos \phi} \left(\sqrt{\frac{8F_1^2 \cos^3 \phi}{1+2K \tan \phi} + 1} - 1 \right) \tag{5}$$

All symbols have been referred to previously, except for the coefficient K, a dimensionless parameter called the shape factor, which varies with the Froude number and the slope of the apron. Kindsvater and Hickox evaluated this coefficient from the profile of the jump and the measured floor pressures. Surface profiles and pressures were not measured in the current tests but, as a matter of interest, K was computed from Expression 5 by substituting experimental values and solving for K. The resulting values of K are listed in Column 17 of Table 8, and are shown plotted with respect to the Froude number for the various slopes on Figure 34A. Superimposed on Figure 34A are data from Kindsvater for a slope of 0.167, and data from Hickox on a slope of 0.333. The agreement is not particularly striking nor do the points plot well, but it should be remembered that the value K is dependent on the method used for determining the length of jump. The current experiments indicate that the Froude number has little effect on the value of K. Assuming this to be the case, values of individual points for each slope were averaged and K is shown plotted with respect to tan ϕ on Figure 34B. This phase is incidental to the study at hand and has been discussed only as a matter of record.

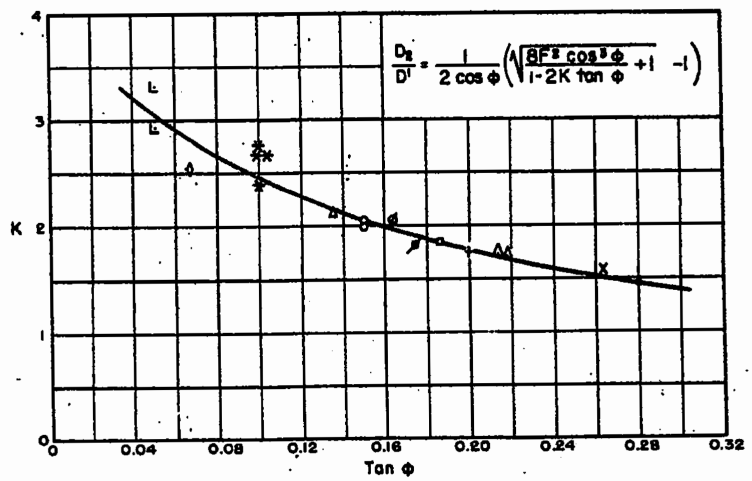
Jump Characteristics (Case B)

Case B is the one usually encountered in sloping apron design where the jump forms both on the slope and over the horizontal portion of the apron (Figure 30B). Although this form of jump may appear quite complicated, it can be readily analyzed when approached from a practical standpoint. The primary concern in sloping apron design is the tail water depth required to move the front of the jump up the slope to Section 1, Figure 30B. There is little to be gained with a sloping apron unless the entire length of the sloping portion is utilized.

Referring to the sketches on Figure 35A, it can be observed that for a tail water equal to the conjugate depth, D_2, the front of the jump will occur at a point 0, a short distance up the slope. This distance is noted as l_o and varies with the degree of slope. If the tail water depth is increased a vertical increment, ΔY_1, it would be reasonable to assume that the front of the jump would raise a corresponding increment. This is not true, the jump profile undergoes an



A



Above curve is based on assumption that K is independent of F_1

B

SYMBOL	Tan φ
	0.052
	.067
	.100
	.135
	.150
	.164
	.174
	.185
	.200
	.215
	.263
	.280
	.167 Kindsvater
	.333 Hickox

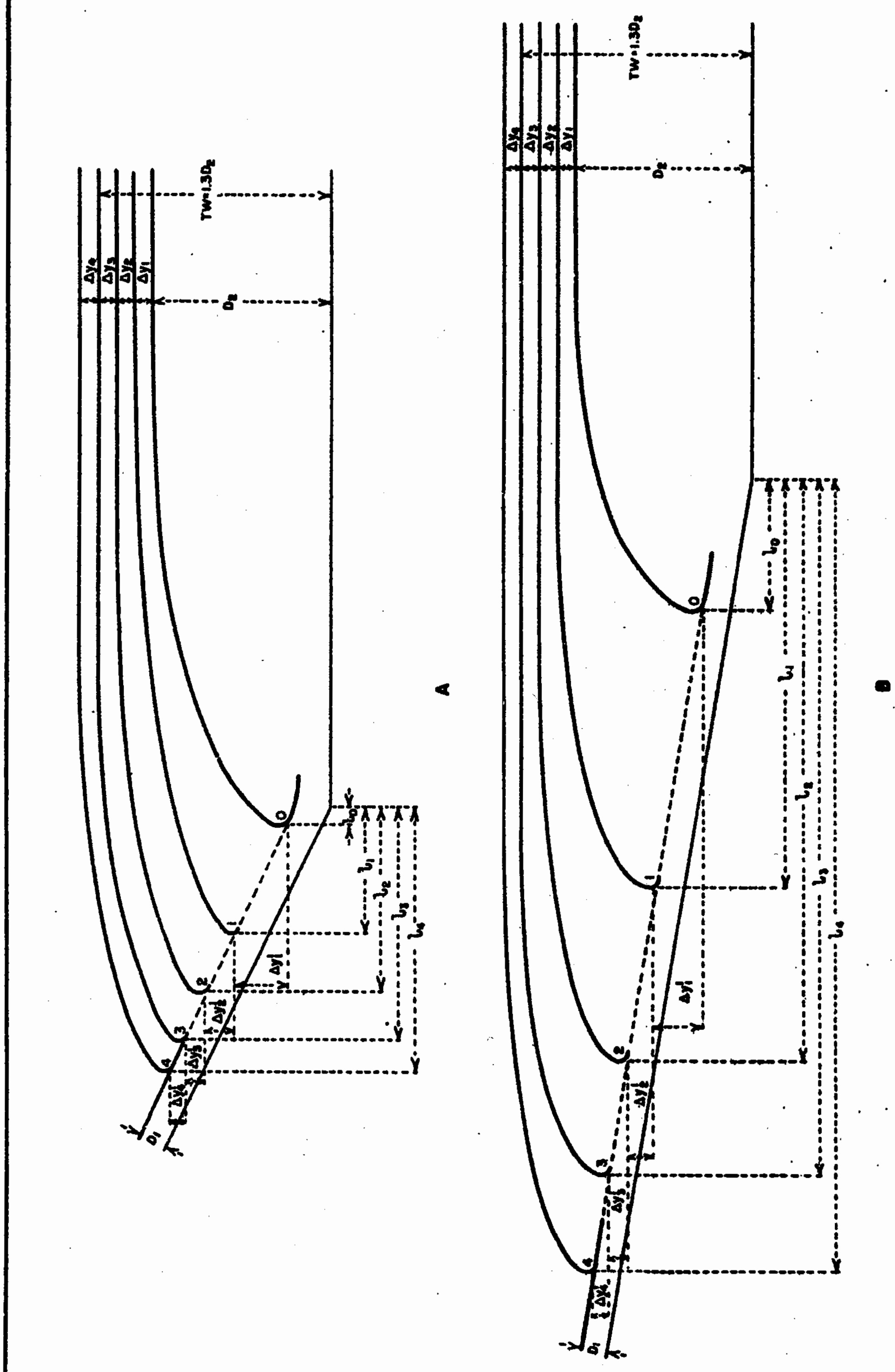
HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE D)

SHAPE FACTOR K

437

FIGURE 35



HYDRAULIC JUMP STUDIES
STILLING BASIN V (CASE B)
PROFILE CHARACTERISTICS

Table 9

STILLING BASINS WITH SLOPING APRONS
(Case B, Basin V)

Test flume (1)	Slope of apron: tan ϕ (2)	Q Total discharge cfs (3)	W Width of basin ft (4)	q Discharge per foot of basin cfs (5)	D_1 Depth entering jump ft (6)	V_1 Velocity entering jump ft/sec (7)	$F_1 = \frac{V_1}{\sqrt{gD_1}}$ (8)	$\frac{D_2}{D_1}$ (9)	D_2 Conjugate TW depth ft (10)	l Length of jump on slope ft (11)	TW Tail-water depth ft (12)	$\frac{l}{D_2}$ (13)	$\frac{TW}{D_2}$ (14)	L_s Length of sloping floor ft (15)
D	0.05	5.050	3.970	1.272	0.063	20.19	14.18	19.51	1.229	6.00	1.390	4.88	1.13	4.0
		8.070		2.033	.101	20.13	11.16	15.30	1.545	6.00	1.745	3.88	1.13	
		11.555		2.910	.139	20.94	9.90	13.60	1.890	6.00	2.040	3.17	1.08	
	.10	5.255		1.324	.067	19.76	13.46	18.60	1.246	4.80	1.440	3.85	1.16	4.0
		8.090		2.038	.103	19.79	10.87	15.00	1.545	4.80	1.750	3.11	1.13	
		11.560		2.911	.140	20.79	9.80	13.40	1.876	4.80	2.080	2.56	1.11	
		5.000		1.259	.064	19.67	13.70	18.90	1.210	8.10	1.830	6.69	1.51	8.0
					.065	19.37	13.38	18.40	1.196	6.30	1.660	5.26	1.39	
					.067	18.79	12.80	17.65	1.183	4.70	1.510	3.97	1.28	
					.068	18.51	12.50	17.20	1.169	4.00	1.410	3.42	1.21	
					.070	17.98	11.99	16.50	1.155	3.20	1.340	2.77	1.16	
		7.850		1.977	.101	19.57	10.86	15.00	1.515	7.80	2.070	5.15	1.37	
					.102	19.38	10.70	14.70	1.499	6.00	1.940	4.00	1.29	
					.103	19.19	10.54	14.50	1.494	5.30	1.880	3.55	1.26	
					.104	19.01	10.39	14.25	1.482	4.40	1.770	2.96	1.19	
		11.218		2.825	.139	20.32	9.61	13.15	1.828	8.30	2.410	4.54	1.32	
					.141	20.04	9.41	12.88	1.816	6.20	2.260	3.41	1.24	
					.142	19.89	9.30	12.80	1.818	4.80	2.180	2.64	1.20	
		6.000		1.511	.076	19.88	12.70	17.50	1.330	2.20	1.375	1.65	1.03	2.0
								17.50	1.330	1.70	1.340	1.28	1.01	
					.077	19.62	12.46	17.15	1.321	0.80	1.305	0.61	0.99	--
					.078	19.37	12.23	16.80	1.310	0	1.280	0	0.98	
		8.057		2.029	.098	20.70	11.66	16.00	1.568	2.40	1.625	1.53	1.04	
					.099	20.49	11.48	15.80	1.564	1.90	1.600	1.22	1.02	
					.100	20.29	11.31	15.60	1.560	1.60	1.585	1.02	1.02	
					.101	20.09	11.14	15.40	1.555	0.60	1.550	0.39	1.00	
					.102	19.89	10.98	15.15	1.545	0	1.530	0	0.99	
	.15	6.000		1.511	.075	20.15	12.96	17.85	1.339	0.50	1.335	0.37	1.00	1.2
								17.85	1.339	1.10	1.365	0.82	1.02	
		8.057		2.029	.099	20.49	11.48	15.80	1.564	0.60	1.564	0.38	1.00	
								15.80	1.564	1.20	1.600	0.77	1.02	--
		11.535		2.905	.136	21.36	10.21	14.00	1.904	0.50	1.905	0.26	1.00	
								14.00	1.904	1.50	1.970	0.79	1.03	
		5.295		1.333	.069	19.32	12.97	17.85	1.232	4.00	1.530	3.25	1.24	
		8.080		2.035	.104	19.57	10.70	14.70	1.529	4.20	1.810	2.75	1.18	
		11.553		2.910	.141	20.64	9.69	13.25	1.868	4.20	2.150	2.25	1.15	
		4.976		1.253	.064	19.58	13.64	18.75	1.200	5.30	1.660	4.42	1.38	5.3
					.065	19.27	13.32	18.35	1.193	5.10	1.590	4.27	1.33	
					.066	18.98	13.02	18.00	1.188	4.00	1.505	3.37	1.27	
					.067	18.70	12.74	17.55	1.176	3.10	1.420	2.64	1.21	
								17.55	1.176	2.60	1.375	2.21	1.17	
					.068	18.42	12.45	17.15	1.166	2.20	1.305	1.89	1.12	
								17.15	1.166	1.80	1.230	1.54	1.05	
		8.025		2.021	.103	19.62	10.77	14.85	1.530	5.30	1.940	3.46	1.27	
								14.85	1.530	4.30	1.875	2.81	1.23	
								14.85	1.530	3.80	1.800	2.48	1.18	
					.104	19.43	10.62	14.60	1.518	2.80	1.705	1.84	1.12	
								14.60	1.518	2.20	1.640	1.45	1.08	
					.105	19.25	10.47	14.40	1.512	1.20	1.580	0.79	1.05	
		11.530		2.904	.142	20.45	9.57	13.10	1.860	5.30	2.260	2.85	1.22	
								13.10	1.860	4.30	2.190	2.31	1.18	
								13.10	1.860	3.60	2.120	1.94	1.14	

Table 9--Continued

STILLING BASINS WITH SLOPING APRONS
(Case B, Basin V)

Test flume	Slope of apron tan ϕ	Q Total discharge cfs	W Width of basin ft	q Discharge per foot of basin cfs	D_1 Depth entering jump ft	V_1 Velocity entering jump ft/sec	$F_1 = \frac{V_1}{\sqrt{gD_1}}$	$\frac{D_2}{D_1}$	D_2 Conjugate TW depth ft	L Length of jump on slope ft	TW Tail-water depth ft	$\frac{L}{D_2}$	$\frac{TW}{D_2}$	L_s Length of sloping floor ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
D	0.29	5.393	3.970	1.358	0.071	19.13	12.65	17.35	1.232	4.60	1.790	3.73	1.45	5.3
								17.35	1.232	4.40	1.720	3.58	1.40	
								17.35	1.232	4.00	1.680	3.25	1.36	
					.072	18.86	12.40	17.05	1.228	3.60	1.605	2.93	1.31	
								17.05	1.228	3.08	1.550	2.44	1.26	
								17.05	1.228	2.60	1.490	2.12	1.21	
					.073	18.60	12.13	16.65	1.212	2.30	1.420	1.90	1.17	
								16.60	1.215	1.50	1.350	1.24	1.11	
								16.60	1.215	1.20	1.280	0.99	1.05	
		8.080		2.035	.105	19.38	10.54	14.50	1.523	4.60	2.010	3.02	1.32	
								14.50	1.523	4.00	1.955	2.63	1.28	
								14.50	1.523	3.30	1.890	2.17	1.24	
					.104	19.57	10.70	14.70	1.529	3.10	1.830	2.03	1.20	
								14.70	1.529	2.50	1.730	1.64	1.13	
								14.70	1.529	1.80	1.670	1.18	1.09	
		11.573		2.915	.145	20.10	9.30	12.80	1.856	4.40	2.310	2.37	1.24	
								12.80	1.856	3.70	2.230	1.99	1.20	
								12.80	1.856	3.30	2.175	1.78	1.17	
		4.820		1.214	.063	19.27	13.53	18.70	1.178	3.70	1.605	3.14	1.36	4.0
		8.089		2.037	.105	19.40	10.55	14.50	1.523	3.90	1.900	2.56	1.25	
		11.565		2.913	.143	20.37	9.50	13.05	1.866	3.90	2.180	2.09	1.17	
	0.25	5.344		1.346	.071	18.96	12.54	17.25	1.225	4.20	1.825	3.43	1.49	5.3
								17.25	1.225	4.10	1.755	3.35	1.43	
								17.25	1.225	3.50	1.680	2.86	1.37	
					.070	19.22	12.81	17.75	1.242	3.00	1.600	2.42	1.29	
								17.75	1.242	2.60	1.525	2.09	1.23	
								17.75	1.242	2.20	1.445	1.77	1.16	
								17.75	1.242	1.60	1.375	1.29	1.11	
								17.75	1.242	0.90	1.290	0.72	1.04	
		8.080		2.035	.107	19.02	10.25	14.05	1.503	4.30	2.100	2.86	1.40	
					.106	19.20	10.40	14.30	1.516	3.40	1.960	2.24	1.29	
								14.30	1.516	2.90	1.860	1.91	1.23	
								14.30	1.516	2.10	1.740	1.38	1.15	
					.105	19.38	10.54	14.50	1.523	1.30	1.650	0.85	1.08	
								14.50	1.523	0.50	1.550	0.33	1.02	
		11.553		2.910	.147	19.80	9.10	12.45	1.830	4.30	2.410	2.35	1.32	
					.146	19.93	9.20	12.60	1.840	4.10	2.320	2.22	1.26	
									1.840	3.20	2.230	1.74	1.21	
					.145	20.07	9.29	12.75	1.849	2.30	2.140	1.24	1.16	
									1.849	1.60	2.090	0.87	1.13	
					.144	20.21	9.39	12.85	1.850	1.20	2.010	0.65	1.09	
									1.850	0.80	1.950	0.43	1.05	
		6.005		1.512	.079	19.14	12.00	16.50	1.306	3.60	1.760	2.76	1.35	4.0
		8.057		2.029	.105	19.32	10.51	14.45	1.517	3.60	1.925	2.37	1.27	
		11.533		2.905	.144	20.17	9.37	12.85	1.850	3.60	2.210	1.95	1.19	
	0.30	8.105		2.041	.110	18.55	9.86	13.50	1.485	4.50	2.300	3.03	1.55	5.3
		5.410		1.362	.074	18.40	11.92	16.40	1.214	4.50	2.070	3.71	1.71	
		11.553		2.910	.150	19.40	8.83	12.05	1.808	4.50	2.570	2.49	1.42	
		5.980		1.506	.079	19.06	11.95	16.45	1.300	3.40	1.840	2.62	1.42	4.0
		8.050		2.028	.106	19.13	10.36	14.25	1.510	3.40	2.025	2.25	1.34	
		11.538		2.906	.146	19.90	9.18	12.55	1.832	3.40	2.300	1.85	1.26	

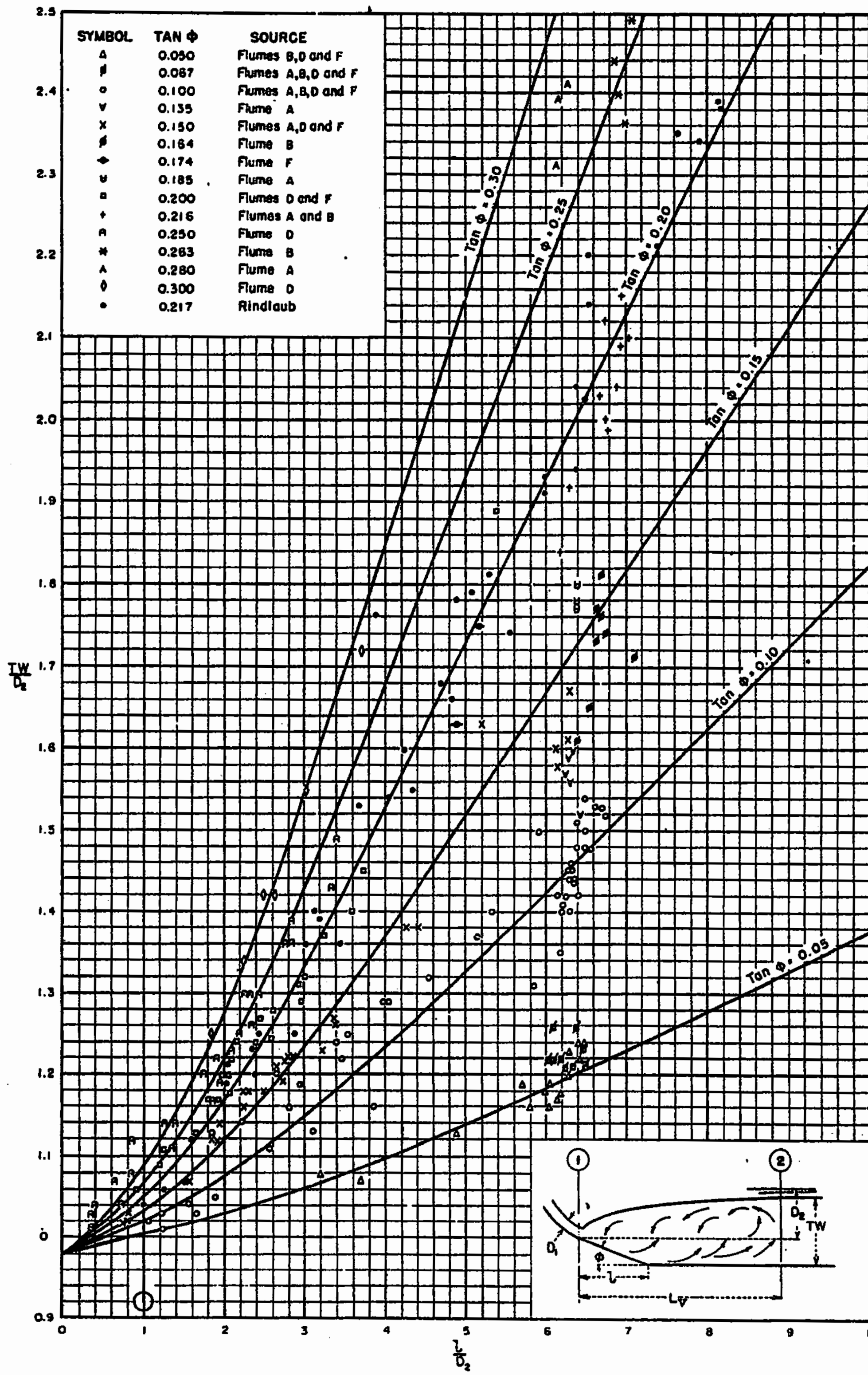
frictional resistance on the slope. The velocity, V_1, and the Froude number were computed at the same location. The tests were made for slopes with tangents varying from 0.05 to 0.30, and in some cases, several lengths of floor were used for each slope, as indicated in Column 15 of Table 9.

The resulting lengths and tail water depths, divided by the conjugate depth, are shown in Columns 13 and 14 of Table 9, and these values have been plotted on Figure 36. The horizontal length has been used rather than the vertical distance, ΔY, as the former dimension is more convenient for use. Figure 36 shows that the straight lines for the geometric portion of the graph tend to intersect at a common point, $\frac{l}{D_2} = 1$ and $\frac{TW}{D_2} = 0.92$, indicated by the circle on the graph. The change in the profile of the jump as it moves from a horizontal floor to the slope is evidenced by the curved portion of the lines.

Case C, Figure 30, is the upper extreme of Case B; and as there is practically no difference in the performance for Cases D and C, data for Case D (Table 8) can again be utilized. By assuming that a horizontal floor begins at the end of the jump in Case D, Columns 15 and 16 of Table 8 can be plotted on Figure 36. In addition, data from experiments by D. D. Rindlaub of the University of California, for a slope of 0.217, have been plotted on Figure 36. The agreement of the information from the three sources is very satisfactory.

Length of Jump (Case B)

It is suggested that the length of jump for Case B be obtained from Figure 33. Actually, Figure 33 is for continuous sloping aprons, but these lengths can be applied to Case B with but negligible error. In some cases the length of jump is not of particular concern because it may not be economically possible to design the basin to confine the entire jump. This is especially true when sloping aprons are used in conjunction with medium or high overfall spillways where the rock in the riverbed is in fairly good condition. When sloping aprons are designed shorter than the length indicated on Figure 33, the rock in the river downstream must act as part of the stilling basin. On the other hand, when the quality of foundation material is questionable, it is advisable to make the apron sufficiently long to confine the entire jump, Figure 33.



HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE B)

TAILWATER REQUIREMENT FOR SLOPING APRONS

PRACTICAL APPLICATIONS

Existing Structures

To determine the practical value of the methods given for the design of sloping aprons, existing basins employing sloping aprons were, in effect, redesigned using the current experimental information. Pertinent data for 13 existing spillways are tabulated in Table 10. The slope of the spillway face is listed in Column 3; the tangent of the sloping stilling basin apron is listed in Column 4; the elevation of the upstream end of the apron, or front of the jump, is listed in Column 7; the elevation of the end of the apron is listed in Column 8; the fall from headwater to upstream end of apron is tabulated in Column 9; and the total discharge is shown in Column 11. Where outlets discharge into the spillway stilling basin, that discharge has also been included in the total. The length of the sloping portion of the apron is given in Column 14; the length of the horizontal portion of the apron is given in Column 15; and the overall length is given in Column 16. Columns 17 through 27 are computations similar to those performed in the previous table.

The lower portions of the curves of Figure 36 have been reproduced to a larger scale on Figure 37. The coordinates from Columns 26 and 27 of Table 10 have been plotted on Figure 37 for each of the 13 spillways. Cross sections of the basins are shown on Figures 38 and 39. Taking the stilling basins in the order shown on Figure 37, we find that the basin apron is not completely utilized for the maximum discharge condition at the Shasta Dam. This discharge includes both spillway and outlet works. The tail water depth is more than sufficient for the jump to utilize the entire stilling basin apron at Capilano Dam; and the full apron length is utilized at Friant, Madden, and Norris Dams spillways. The entire apron length will not be utilized for the maximum discharge at Canyon Ferry Dam. In this case the apron was designed for a discharge of 200,000 cfs but the stilling basin will operate at 250,000 cfs without sweeping out. Keswick shows a deficiency in tail water depth for utilization of the entire apron, but this is compensated for, to some extent, by large spreader teeth at the upstream end of the apron. For the preliminary and final basin designs for the Bhakra Dam spillway, both utilize practically the full length of apron. The jump will not occupy the full length of apron for maximum discharge on Olympus, Folsom, or Rihand Dams spillways. The jump will form downstream from the upstream end of the slope. The models of the latter two structures actually showed this to be true. The full length of apron will be utilized by the jump for the stilling basin at Dickinson Dam. This was an earth dam spillway in which appurtenances were used in the basin.

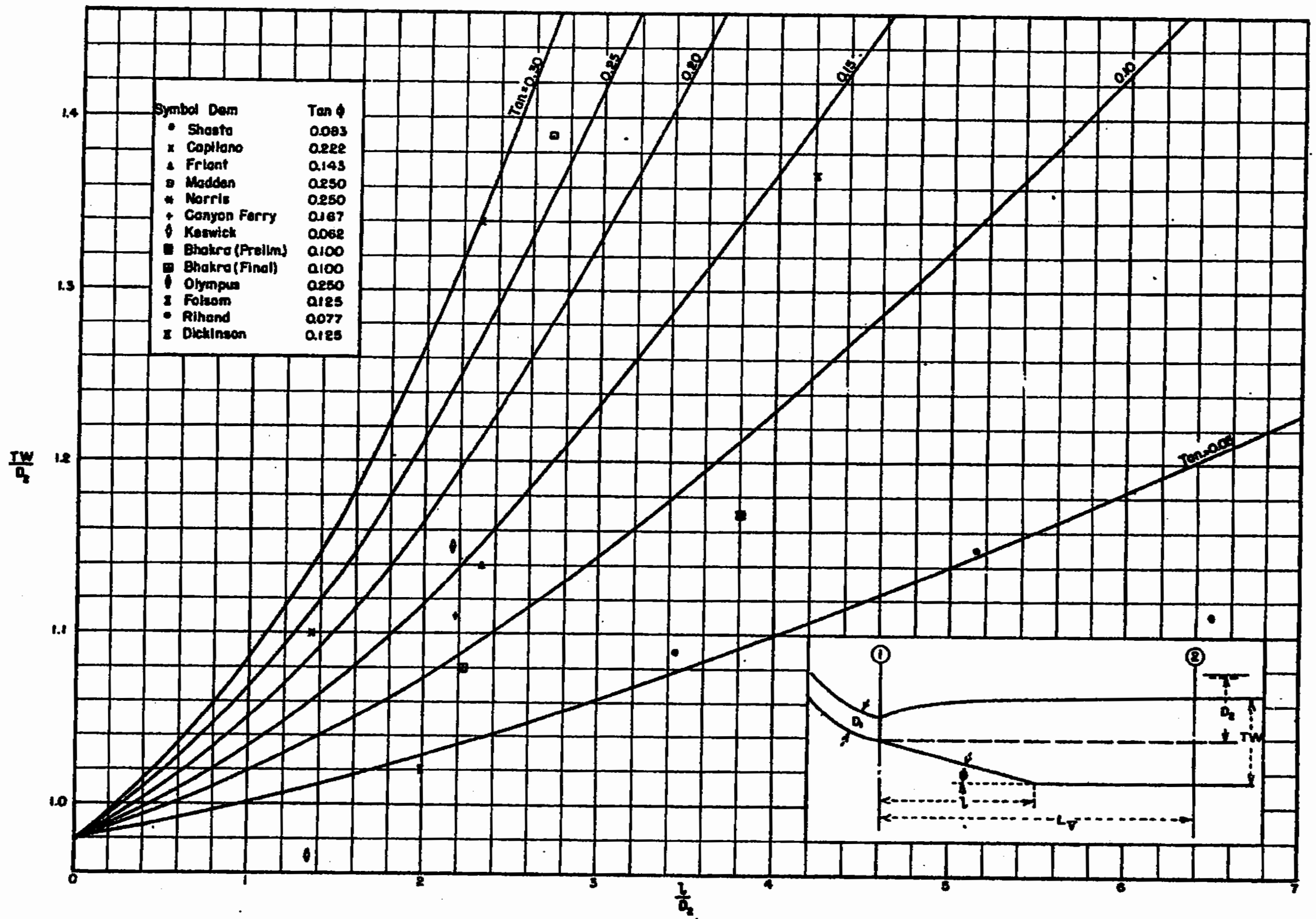
Table 10

EXISTING STILLING BASINS WITH SLOPING APRONS

Dam	Location	Slope of dam face	Slope of stilling basin apron tan ϕ	Reservoir elevation ft	Crest elevation ft	Elevation upstream end of apron ft	Elevation downstream end of apron ft	Fall headwater to U.S. end of apron ft	Head on crest of spillway ft	Q Max discharge cfs	Max tail-water elevation ft	TW depth ft	l Length of sloping apron ft	h Length of horizontal apron ft	l + h Total length of apron ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Shasta	California	0.8:1	0.083	1065	1037	570.8	549.5	494.4	28.0	250,000	631.0	81.5	256.7	51.9	308.6
Norris	Tennessee	0.7:1	.250	1047	1020	826.0	805.5	221.0	27.0	197,600	872.0	66.5	81.5	142.5	224
Bhakra (prelim)	India	0.8:1	.100	1580	1552	1139.4	1112.2	440.6	28.0	189,600	1196.0	83.8	257	15	272
Canyon Ferry	Montana	Varies	.167	3800	3766	3625.0	3600.0	175.0	34.0	200,000	3670.0	70.0	137	57	194
Bhakra (final)	India	0.8:1	.100	1685	1645	1117.5	1095.0	567.5	40.0	290,000	1205.0	110.0	224.5	165	389.5
Madden	Canal Zone	0.75:1	.250	250	232	98.0	64.5	152.0	18.0	280,000	141.5	77.0	142	8	150
Folsom	California	0.67:1	.125	466	418	137.0	115.0	329.0	48.0	250,000	205.5	90.5	177	147	324
Olympus	Colorado	Varies	.250	7475	7460	7417.0	7405.0	58.0	15.	20,000	7431.0	26.0	48.5	43.6	92.1
Capilano	British Colombia	0.65:1	.222	570	547	274.0	246.0	296.0	23.0	43,000	320.0	74.0	*128	106	234
Rihand	India	0.7:1	.077	888	852	647.6	604.0	240.4	36.0	455,000	679.0	75.0	325	10	335
Friant	California	0.7:1	.143	578	560	296.0	282.5	282.0	18.0	90,000	330.0	47.5	97	125	222
Keswick	California	Varies	.062	587	537	488.6	483.8	98.4	50.0	250,000	541.0	57.2	105	23	128
Dickinson	North Dakota	0.5:1	.125	2428	2416	2388.0	2380.0	40.9	12.4	33,200	2404.7	23.0	61.5	9.5	71

Dam	V_{th} Theoretical velocity entering basin ft/sec	$\frac{V_A}{V_{th}}$	$V_A = V_1$ Actual velocity entering basin ft/sec	W Width of basin ft	q Discharge per foot of basin cfs	D_1 Depth entering basin ft	$F_1 = \frac{V_1}{\sqrt{gD_1}}$	$\frac{D_2}{D_1}$	D_2 Conjugate TW depth ft	$\frac{l}{D_2}$	$\frac{TW}{D_2}$	$\frac{L}{D_2}$	L Actual length of jump ft	$\frac{l + h}{L}$
	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)
Shasta	176	0.81	141	375	667	4.73	11.42	15.75	74.5	3.44	1.09	6.30	469	0.66
Norris	116	0.91	106	332	595	5.61	7.88	10.70	60.0	1.36	1.11	7.03	422	.53
Bhakra (prelim)	166	0.82	136	300	632	4.65	11.11	15.25	70.9	3.63	1.18	6.35	450	.60
Canyon Ferry	101	0.95	96	271	738	7.69	6.10	8.20	63.1	2.17	1.11	6.55	413	.47
Bhakra (final)	188	0.85	158	260	1115	7.15	10.27	14.20	101.5	2.21	1.08	6.36	646	.60
Madden	96	0.91	87	448	625	7.18	5.72	7.70	55.3	2.57	1.39	6.90	382	.39
Folsom	140	0.95	133	242	1033	7.76	8.41	11.45	88.9	1.99	1.02	6.50	578	.56
Olympus	57	0.95	54	120	167	3.09	5.41	7.30	22.6	2.15	1.15	6.86	155	.59
Capilano	135	0.87	117	80	538	4.60	9.62	13.10	60.3	2.12	1.23	6.85	384	.61
Rihand	117	0.92	108	664	685	6.34	7.56	10.25	65.0	5.00	1.15	6.30	413	.82
Friant	133	0.81	108	330	273	2.53	11.97	16.45	41.6	2.33	1.14	6.40	266	.83
Keswick	69	1.00	69	240	1042	15.15	3.12	4.00	60.6	1.73	0.94	5.45	330	.39
Dickinson	51	--	48	200	166	3.50	4.44	5.70	20.0	3.08	1.19	6.00	120	.59

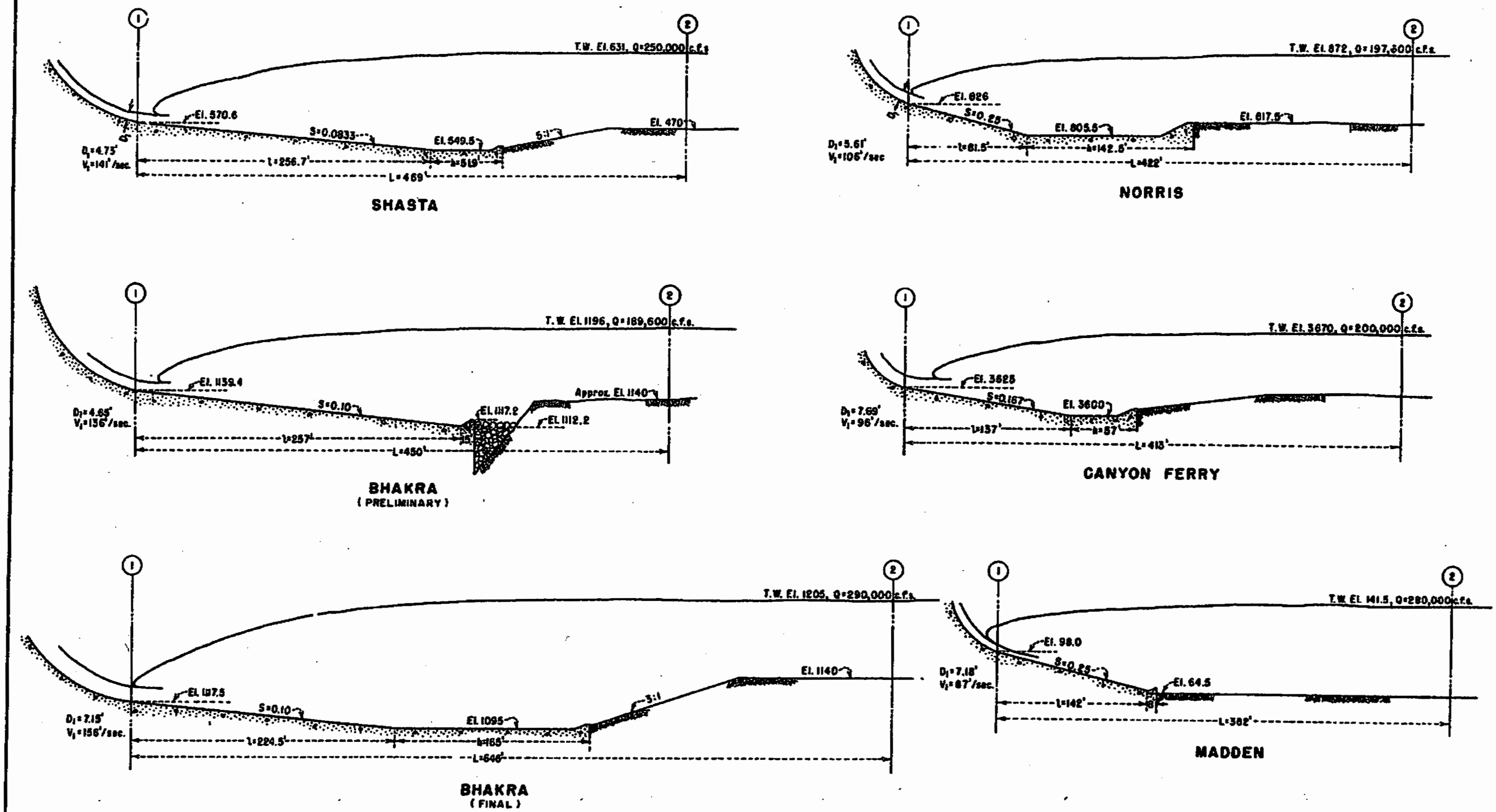
Average 0.60



HYDRAULIC JUMP STUDIES

STILLING BASIN V (CASE B)

COMPARISON OF EXISTING SLOPING APRON DESIGNS WITH CURRENT EXPERIMENTAL RESULTS



HYDRAULIC JUMP STUDIES
STILLING BASIN V (CASE B)
EXISTING BASINS WITH SLOPING APRONS
SHEET 1 OF 2

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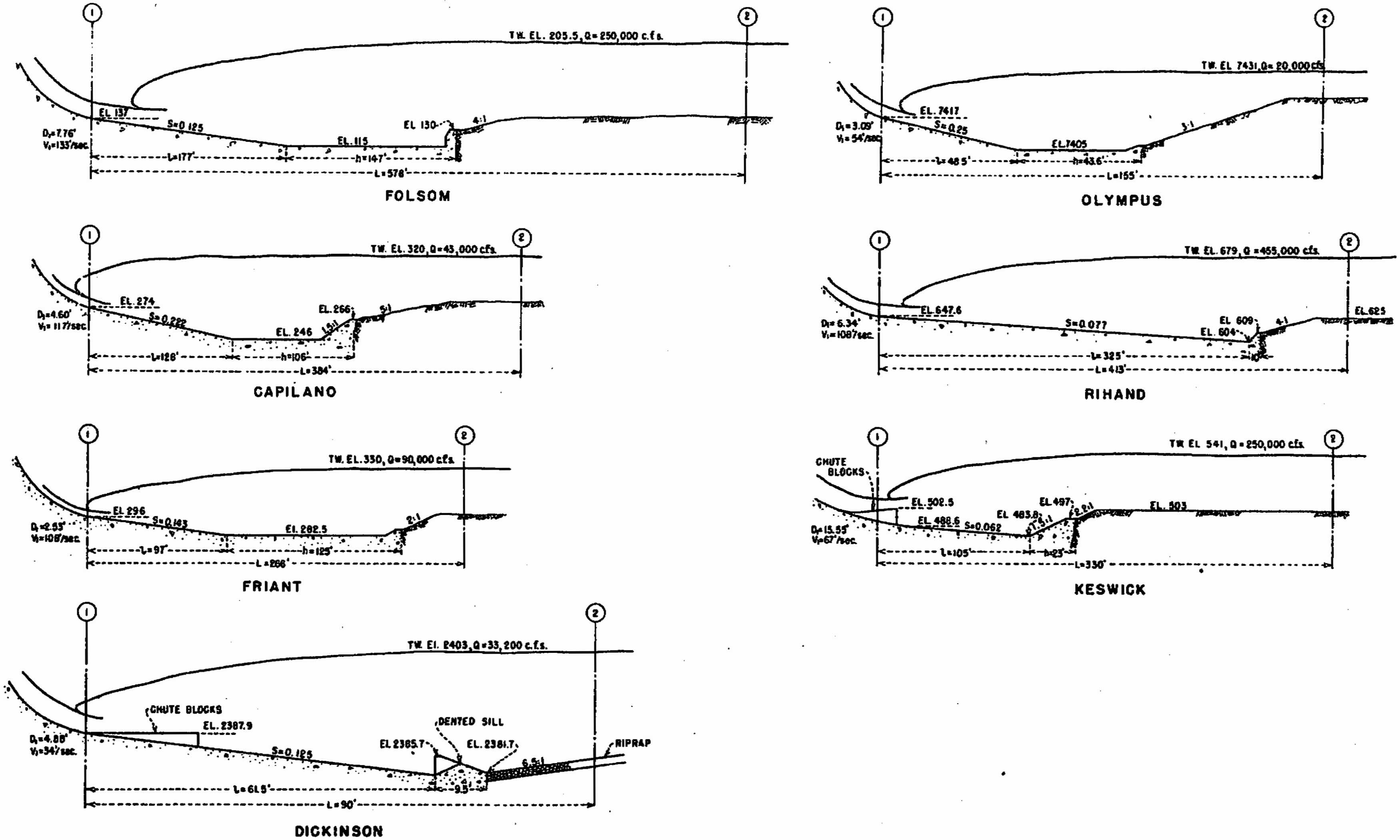
FIGURE 39



HYDRAULIC JUMP STUDIES
STILLING BASIN V (CASE B)
EXISTING BASINS WITH SLOPING APRONS
SHEET 2 OF 2

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All of the structures listed in Table 10 and shown on Figures 38 and 39 were designed with the aid of model studies. The degree of conservatism used in each case was dependent on local conditions and the individual designer.

The total lengths of apron provided for the above 13 existing structures are shown in Column 16 of Table 10. The length of jump for the maximum discharge condition for each case is tabulated in Column 29 of the same table. The ratio of total length of apron to length of jump is shown in Column 30. The total apron length ranges from 39 to 83 percent of the length of jump; or considering the 13 structures collectively, the average total length of apron is 60 percent of the length of the jump.

Evaluation of Sloping Aprons

A convincing argument, quoted by the laboratory and others in the past, has been that sloping aprons should be designed so that the jump height curve matches the tail water curve for all discharge conditions. This procedure results in what has been designated a tailor-made basin. Some of the existing basins shown on Figures 38 and 39 were designed in this manner. In light of the current experiments, it was discovered that this course is not the most desirable approach. Instead, matching the jump height curve with the tail water curve should be a secondary consideration, except for the maximum discharge condition.

Thus, the first consideration in design is to determine the apron slope that will involve the minimum amount of excavation, the minimum amount of concrete, or both, for the maximum discharge and tail water condition. This is the prime consideration. Only then is the jump height checked to determine whether the tail water depth is adequate for the intermediate discharges. It will be found that the tail water depth usually exceeds the required jump height for the intermediate discharges. This may result in a slightly submerged condition for intermediate discharges, but performance will be very acceptable. The extra depth will provide a smoother water surface in and downstream from the basin. Should the tail water depth be insufficient for intermediate flows, it will be necessary to increase the depth by increasing the slope, or reverting to a horizontal basin. It is not necessary for the front of the jump to form at the upstream end of the sloping apron for intermediate discharges provided the tail water depth and the length of basin available for energy dissipation are considered adequate. Using this method, the designer is free to choose the slope he desires, since tests showed that the slope itself had little effect on the performance of the stilling basin action.

It is not possible to standardize on sloping apron design nearly as much as for the horizontal aprons, as much more individual judgment is required. The slope and overall shape of the apron must be determined from economic reasoning, while the length must be judged by the type and soundness of the riverbed downstream. The existing structures shown on Figures 38 and 39 should serve as a guide in proportioning future sloping apron designs.

Sloping Apron Versus Horizontal Apron

A point, which it is felt has been misunderstood in the past with regard to horizontal aprons for high dams, can now be clarified. The Bureau has constructed very few stilling basins with horizontal aprons for its larger dams. It has been the consensus that the hydraulic jump on a horizontal apron is very sensitive to slight changes in tail water depth. This is very true for the larger values of the Froude number, but this characteristic can be remedied. Suppose a horizontal apron is designed for a Froude number of 10. The basin will operate satisfactorily for conjugate tail water depth, but as the tail water is lowered to $0.98D_2$ the front of the jump will begin to move. By the time the tail water is dropped to $0.96D_2$, the jump will probably be completely out of the basin. Thus, to design a stilling basin in this range the tail water depth must be known with certainty or a factor of safety should be provided in the design. To guard against a deficiency in tail water depth, the same procedure is suggested here as for Basins I and II. Referring to the minimum tail water curve for Basins I and II on Figure 11, the margin of safety can be observed for any value of the Froude number. It is recommended that the tail water depth for maximum discharge be at least 5 percent larger than the minimum shown on Figure 11. For values of the Froude number greater than 9, a 10 percent factor of safety may be advisable as this will not only stabilize the jump but will improve the performance. With the additional tail water depth, the horizontal apron will perform on a par with the sloping apron. Thus, the primary consideration in design need not be hydraulic but structural. The basin, with either horizontal or sloping apron, which can be constructed at the least cost is the most desirable.

Effect of Slope of Chute

A factor which occasionally affects stilling basin operation is the slope of chute entering the basin. The foregoing experimentation was sufficiently extensive to shed some light on this factor. The tests showed that the slope of chute upstream from the stilling basin was unimportant, as far as jump performance was concerned, so long as the velocity distribution in the jet entering the jump was reasonably uniform. In the case of steep chutes or short flat chutes, the velocity distribution can be considered normal. The principal difficulty is experienced with long flat chutes where frictional resistance on bottom and side walls is sufficient to produce a center velocity greatly exceeding that on the bottom or sides. When this happens, greater activity results in the center of the stilling basin than on the sides producing an asymmetrical jump with strong side eddies. This same effect is also witnessed when the angle of divergence of a chute is too great for the water to follow properly. In either case the surface of the jump is unusually rough and choppy and the position of the front of the jump is not always predictable.

In the case of earth dam spillways the practice has been to make the upstream portion unusually flat, then steepen the slope to 2:1, or that corresponding to the natural trajectory of the jet, immediately preceding the stilling basin. Figure 1A, which shows the model spillway for Trenton Dam, illustrates this practice. Bringing an asymmetrical jet into the stilling basin at a steep angle usually does aid in stabilizing the jump. This is not effective, however, where very long flat slopes are involved and the velocity distribution is completely out of balance.

The most adverse condition has been observed where long canal chutes terminate in stilling basins. A typical example is the chute and basin at Station 25+19 on the South Canal, Uncompahgre Project, Colorado, Figure 40. The operation of this stilling basin is not particularly objectionable, but it will serve as an illustration. The above chute is approximately 700 feet long with a slope of 0.0392. The stilling basin at the end is also shown on Figure 40. A photograph of the prototype basin operating at normal capacity is shown on Figure 41. The action is of the surging type; the jump is unusually rough, with a great amount of splash and spray. Two factors contribute to the rough operation: the unbalanced velocity distribution in the entering jet, and excessive divergence of the chute in the steepest portion.

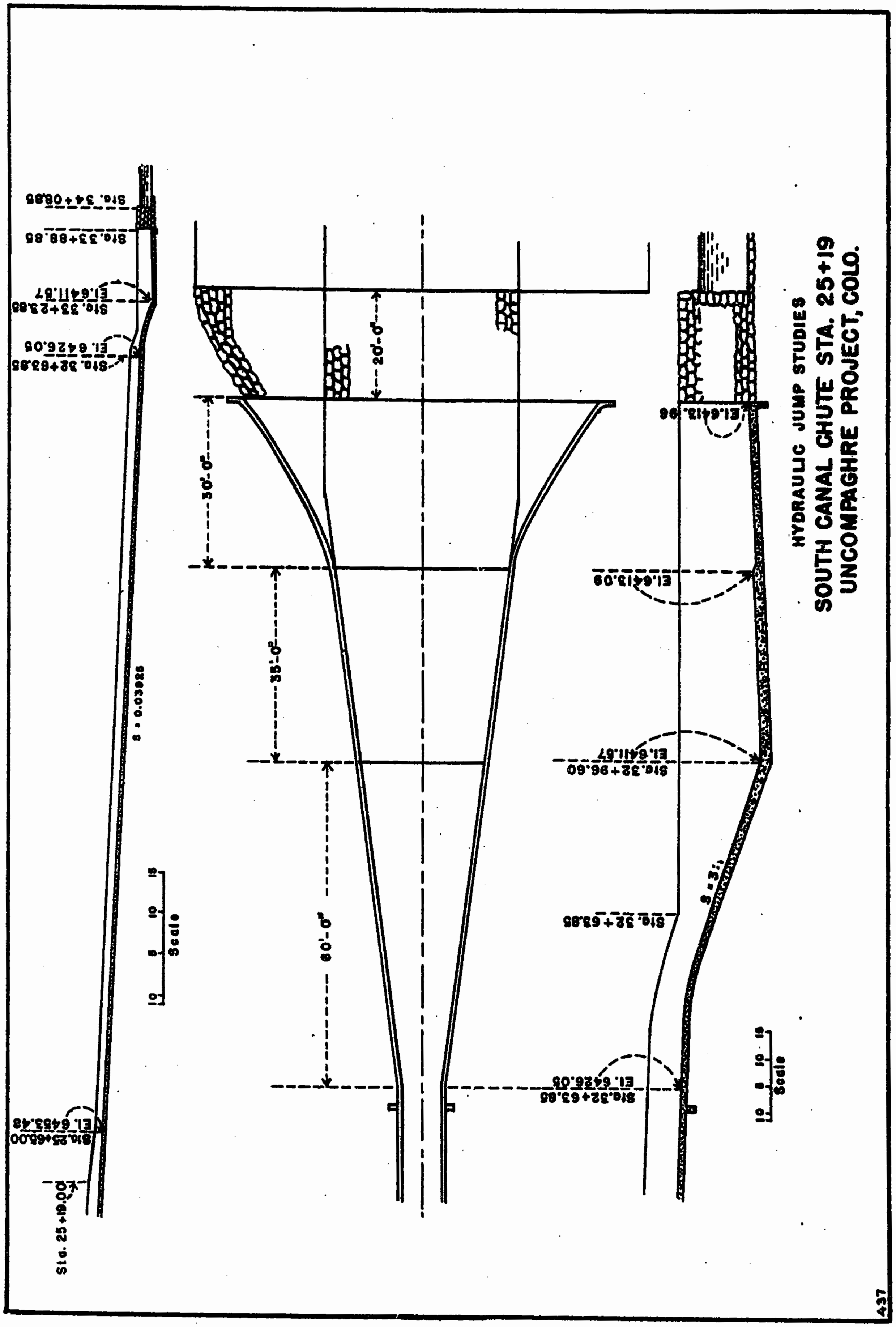
A definite improvement can be accomplished in future designs where long flat chutes are involved by utilizing the Type III basin described in Section 3. The baffle blocks on the floor tend to alter the asymmetrical jet, resulting in an overall improvement in operation. This is the only corrective measure that can be suggested at this time.

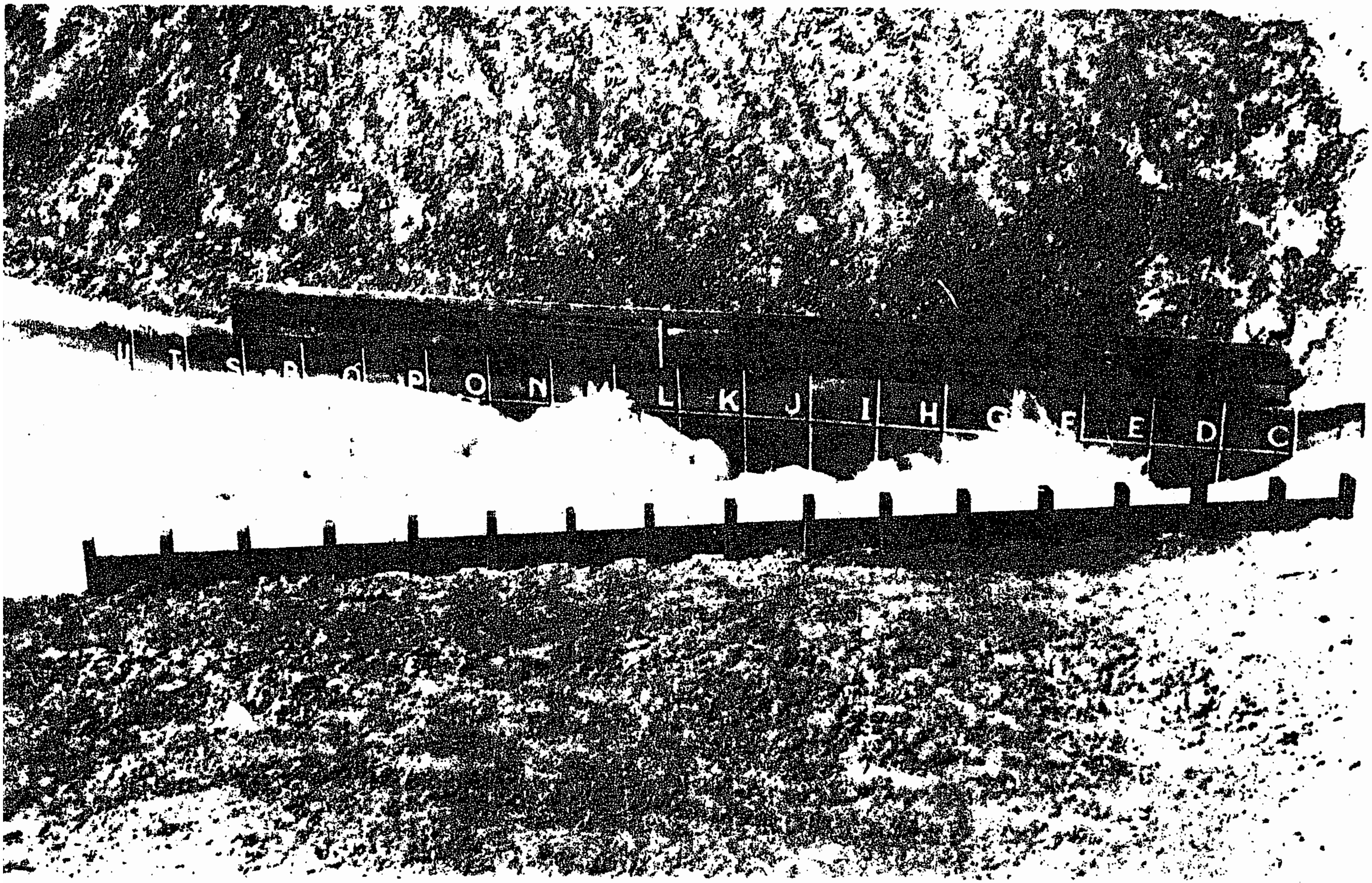
Recommendations

The following rules have been devised for the design of sloping aprons as developed from the foregoing experiments:

1. Determine an apron arrangement which will give the greatest economy for the maximum discharge condition. This is the governing factor and the only justification for using a sloping apron.

FIGURE 40


HYDRAULIC JUMP STUDIES
SOUTH CANAL CHUTE STA. 25+19
UNCOMPAGHRE PROJECT, COLO.
Sta. 34+08.85
Sta. 33+88.85
El. 6411.57
Sta. 33+23.85
El. 6426.05
Sta. 32+63.85
S = 0.03928
Sta. 25+65.00
El. 6453.48
Sta. 25+19.00
20'-0"
30'-0"
35'-0"
60'-0"
El. 6413.96
El. 6413.09
El. 6411.67
Sta. 32+96.60
S = 3:1
Sta. 32+63.85
El. 6426.05
Sta. 32+63.85
Scale
437



CHUTE STILLING BASIN ON SOUTH CANAL
UNCOMPAHGRE PROJECT

2. Position the apron so that the front of the jump will form at the upstream end of the slope for the maximum discharge and tail water condition by means of the information on Figure 37. Several trials will usually be required before the slope and location of the apron are compatible with the hydraulic requirement. It may be necessary to raise or lower the apron, or change the original slope entirely.

3. The length of the jump for maximum or partial flows can be obtained from Figure 33. The portion of the jump to be confined on the stilling basin apron is a decision for the designer. In making this decision, Figures 38 and 39 may be helpful. The average overall apron in Figures 38 and 39 averages 60 percent of the length of jump for the maximum discharge condition. The apron may be lengthened or shortened, depending upon the quality of the rock in the riverbed and other local conditions. If the apron is set on loose material and the downstream channel is in poor condition, it may be advisable to make the total length of apron the same as the length of jump.

4. With the apron designed properly for the maximum discharge condition, the next step is to be certain that the tail water depth and length of basin available for energy dissipation are sufficient for, say, 1/4, 1/2, and 3/4 capacity. If the tail water depth is sufficient or in excess of the jump height for the intermediate discharges, the design is acceptable. If the tail water depth is deficient, it may then be necessary to try a flatter slope or reposition the sloping portion of the apron. It is not necessary that the front of the jump form at the upstream end of the sloping apron for partial flows. In other words, the front of the jump may remain at Section 1 (Figure 30B), move upstream from Section 1, or move down the slope for partial flows, providing the tail water depth and length of the apron are considered sufficient for these flows.

5. A horizontal apron will perform on a par with the sloping apron, for high values of the Froude number, if the proper tail water depth is provided.

6. The slope of the chute upstream from a stilling basin has little effect on the hydraulic jump so long as the velocity distribution and depth of flow are reasonably uniform on entering the jump.

7. A small solid triangular sill, placed at the end of the apron, is the only appurtenance needed in conjunction with the sloping apron. It serves to lift the flow as it leaves the apron and thus acts to control scour. Its dimensions are not critical; the most effective height is between $0.05D_2$ and $0.10D_2$ and a slope of 3:1 to 2:1 (see Figures 38 and 39).

A spillway should be operated to produce as nearly symmetrical flow in the stilling basin as possible. (This applies to all stilling basins.) Asymmetry produces large horizontal eddies that can carry riverbed material onto the apron. This material, motivated by the energy in the eddies, can abrade the apron and appurtenances in the basin at a very surprising rate. These eddies can also undermine wing walls and settle riprap. Asymmetrical operation is expensive operation, and operators should be continually reminded of this fact.

Where the discharge over high spillways exceeds 500 cfs per foot of apron width, or where there is any form of asymmetry involved, a model study is advisable. For the higher values of the Froude number, stilling basins become increasingly expensive, and the performance less acceptable. Thus, where practical, a bucket type of dissipator may serve the purpose better and more economically than a stilling basin.

SECTION 6

STILLING BASIN FOR PIPE OR OPEN CHANNEL OUTLETS
NO TAIL WATER REQUIRED
(BASIN VI)

SUMMARY

The stilling basin developed in these tests is an impact-type energy dissipator, contained in a relatively small boxlike structure, which requires no tail water for successful performance. Although the emphasis in this discussion is placed on use with pipe outlets, the entrance structure may be modified to use an open channel entrance.

Generalized design rules and procedures are presented to allow determining the proper basin size and all critical dimensions for a range of discharges up to 339 feet per second and velocities up to 30 feet per second.* Greater discharges may be handled by constructing multiple units side by side. The efficiency of the basin in accomplishing energy losses is greater than a hydraulic jump of the same Froude number.

INTRODUCTION

The development of this short impact-type basin was initiated by the need for some 50 or more stilling structures on the Franklin Canal, Bostwick Division, Missouri River Basin Project. The need was for relatively small basins providing energy dissipation independent of a tail water curve or tail water of any kind. The demand for information on general design procedures for use on other projects prompted the laboratory to include further investigation of this basin in the laboratory's general research program. Continued research on this type of basin will be made as time and funds permit.

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*The laboratory has developed two basins for specific installations where velocities were considerably higher. One basin was for 10 second-feet at 80 feet per second, the other for 4 second-feet at 106 feet per second (see Bibliography, No. 33). Sufficient data are not available, however, to provide general design rules or procedures.

TEST PROCEDURE

Hydraulic Models

Hydraulic models were used to develop the stilling basin, determine the discharge limitations, and obtain dimensions for the various parts of the basin. Basins 1.6 to 2.0 feet wide were used in the tests. The inlet pipe was 6-3/8 inches, inside diameter, and was equipped with a slide gate well upstream from the basin entrance so that the desired relations between head, depth, and velocity could be obtained. The pipe was transparent so that backwater effects in the pipe could be studied. Discharges of over 3 cubic feet per second and velocities up to 15 feet per second could be obtained during the tests. Hydraulic model-prototype relations were used to scale up the results to predict performance for discharges up to 339 second-feet and velocities up to 30 feet per second.

The basin was tested in a tail box containing gravel formed into a trapezoidal channel. The size of the gravel was changed several times during the tests. The outlet channel bottom was slightly wider than the basin and had 1:1 side slopes. A tail gate was provided at the downstream end to evaluate the effects of tail water.

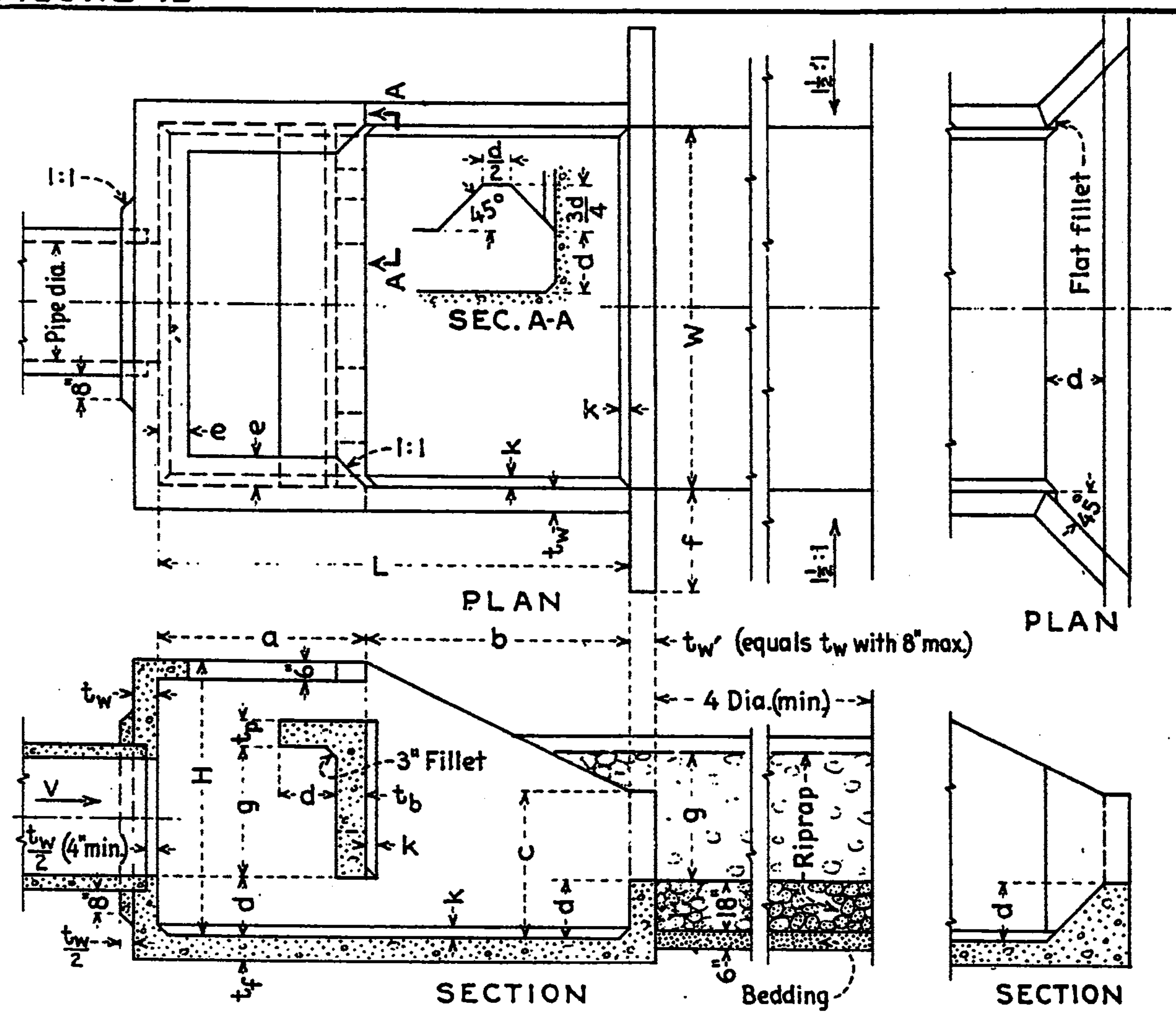
Development of Basin

The finally evolved basin was the result of extensive tests on many different arrangements. A detailed discussion of these tests is not given since they had little if any bearing on the final design except in a general way. This is discussed below.

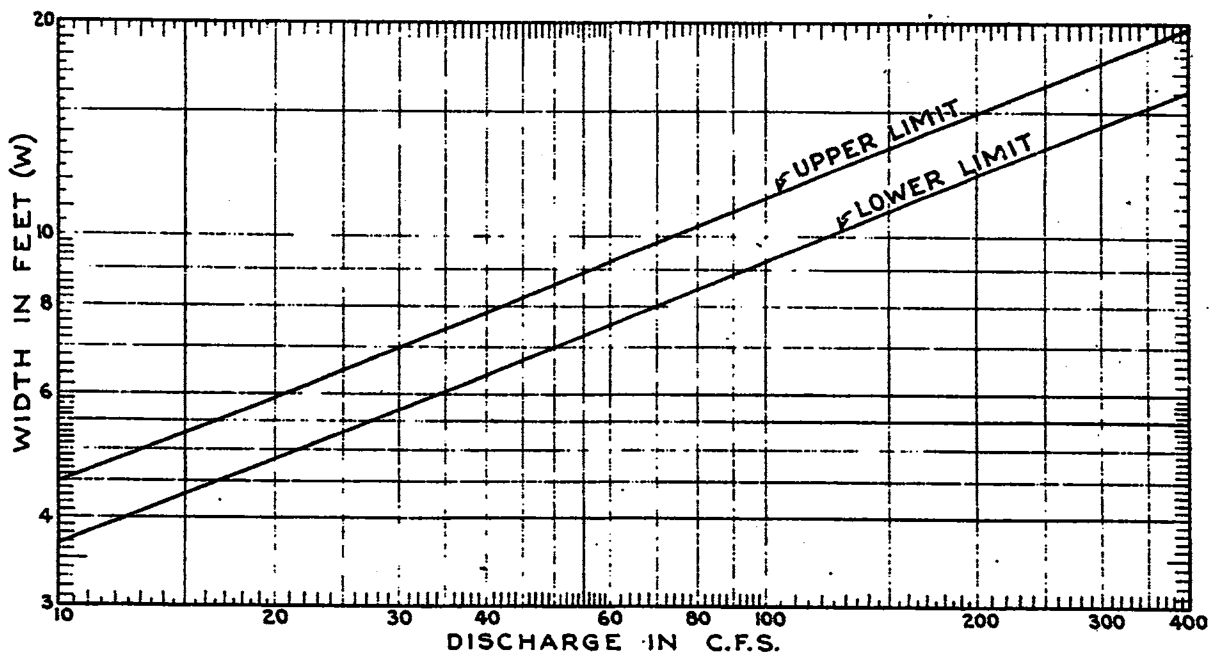
With the many combinations of discharge, velocity, and depth possible for the incoming flow, it became apparent during the early tests that some device was needed at the stilling basin entrance to convert the many possible flow patterns into a common pattern. The vertical hanging baffle proved to be this device, Figure 42. Regardless of the depth or velocity of the incoming flow (within the prescribed limits) the flow after striking the baffle acted the same as any other combination of depth and velocity. Thus, some of the variables were eliminated from the problem.

The effect of velocity alone was then investigated, and it was found that for velocities 30 feet per second and below (for a 42-inch pipe) the performance of the structure was primarily dependent on the discharge. Actually, the velocity of the incoming flow does affect the performance of the basin, but from a practical point of view it could be eliminated from consideration. Had this not been done, considerably more testing would have been required to evaluate and express the effect of velocity.

FIGURE 42



STILLING BASIN DESIGN



DISCHARGE LIMITS

IMPACT TYPE ENERGY DISSIPATOR
(BASIN VI)

For velocities of 30 feet per second or less the basin width W was found to be a function of the discharge, with other basin dimensions being related to the width, Figure 42. To determine the necessary width, erosion test results, judgment, and operating experiences were all used and the advice of laboratory and design personnel was used to obtain the finally determined limits. Since no definite line of demarcation between a "too wide" or "too narrow" basin exists, it was necessary to work between two more definite lines, shown on Figure 42 as the upper and lower limits. These lines required far less judgment to determine than a single intermediate line.

Various basin sizes, discharges, and velocities were tested taking note of the erosion, wave heights, energy losses, and general performance. When the upper and lower limit lines had been established a line about midway between the two was used to establish the proper width of basin for various discharges. The exact line is not shown because strict adherence to a single curve would result in difficult to use fractional dimensions. Accuracy of this degree is not justifiable. Figure 43 shows typical performance of the recommended stilling basin for the three limits discussed. It is evident that the center photograph represents a compromise between the upper limit operation which is very mild and the lower limit operation which is approaching the unsafe range.

Using the middle range of basin widths, other basin dimensions were determined, modified, and made minimum by means of trial and error tests on the several models. Dimensions for nine different basins are shown in Table 11. These should not be arbitrarily reduced since in the interests of economy the dimensions have been reduced as far as is safely possible.

Performance of Basin

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. The structure, therefore, requires no tail water for energy dissipation as is necessary for a hydraulic jump basin. Tail water as high as $d + \frac{b}{2}$, Figure 42, however, will improve the performance by reducing outlet velocities, providing a smooth water surface, and reducing tendencies toward erosion. Excessive tail water, on the other hand, will cause some flow to pass over the top of the baffle. This should be avoided if possible.

The effectiveness of the basin is best illustrated by comparing the energy losses within the structure to those which occur in a hydraulic jump. Based on depth and velocity measurements made in the approach pipe and in the downstream channel (no tail water), the change in momentum was computed as explained in Section 1 for the hydraulic jump.

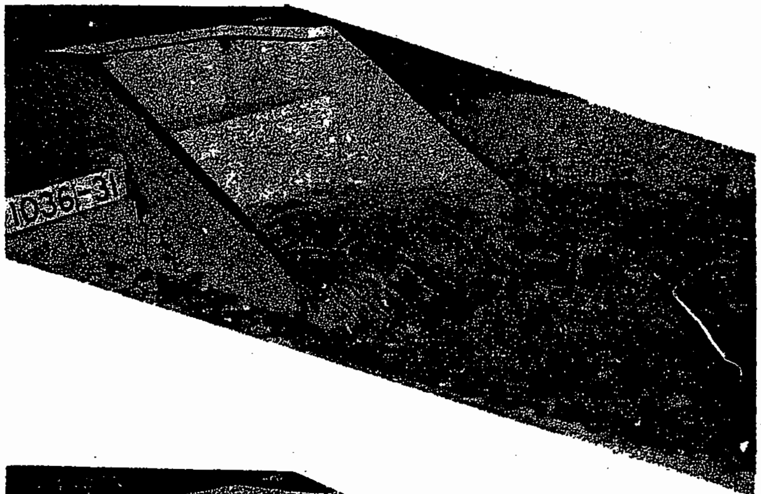
Table 11

STILLING BASIN DIMENSIONS
Impact-type Energy Dissipator
(Basin VI)

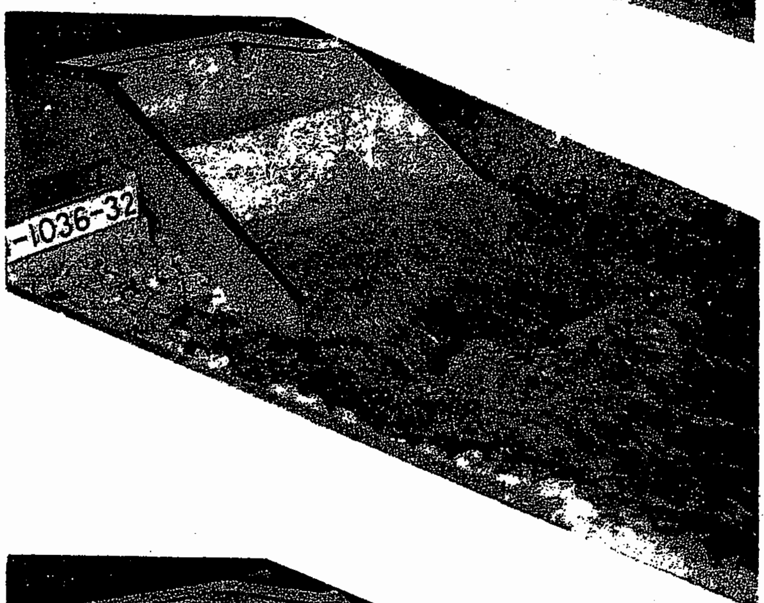
Suggested pipe size*		Maximum discharge	Feet and inches										Inches				
Dia in. (1)	Area (sq ft) (2)	Q (3)	W (4)	H (5)	L (6)	a (7)	b (8)	c (9)	d (10)	e (11)	f (12)	g (13)	t_w (14)	t_f (15)	t_b (16)	t_p (17)	K (18)
18	1.7672	21**	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1	6	6-1/2	6	6	3
24	3.1416	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6	6	6-1/2	6	6	3
30	4.9087	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0	6	6-1/2	7	7	3
36	7.0686	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6	7	7-1/2	8	8	3
42	9.6211	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11	8	8-1/2	9	8	4
48	12.5664	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	4-5	9	9-1/2	10	8	4
54	15.9043	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	1-0	3-0	4-11	10	10-1/2	10	8	4
60	19.6350	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4	11	11-1/2	11	8	6
72	28.2743	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	1-3	3-0	6-2	12	12-1/2	12	8	6

*Suggested pipe will run full when velocity is 12 feet per second or half full when velocity is 24 feet per second. Size may be modified for other velocities by $Q = AV$, but relation between Q and basin dimensions shown must be maintained.

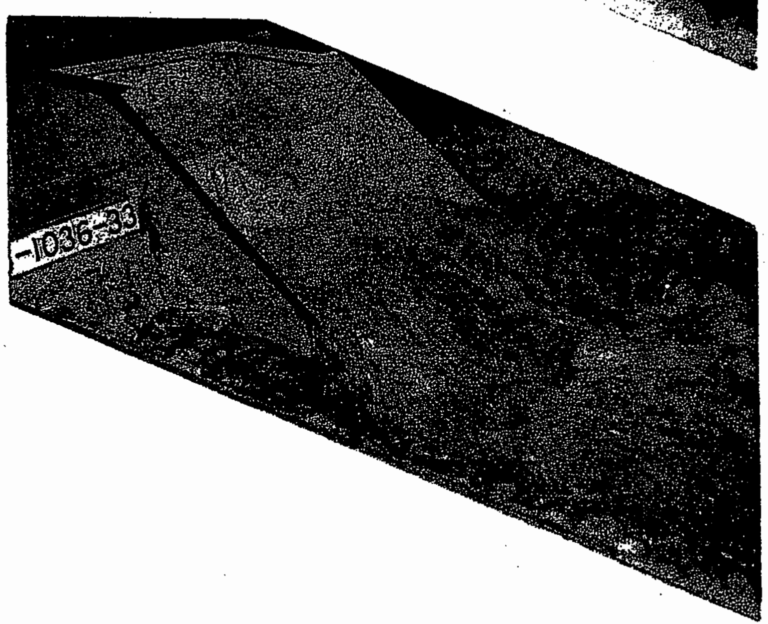
**For discharges less than 21 second-feet, obtain basin width from curve of Figure 42. Other dimensions proportional to W; $H = \frac{3W}{4}$, $L = \frac{4W}{3}$, $d = \frac{W}{6}$, etc.



Lowest value of maximum discharge - Corresponds to upper limit curve



Intermediate value of maximum discharge - Corresponds to tabular values



Largest value of maximum discharge - Corresponds to lower limit curve

Typical Performance at Maximum Discharges - No Tailwater
Impact Type Energy Dissipator - Basin VI

The Froude number of the incoming flow was computed using D_1, obtained by converting the flow area in the partly full pipe into an equivalent rectangle as wide as the pipe diameter. Compared to the losses in the hydraulic jump, Figure 44, the impact basin shows greater efficiency in performance. Inasmuch as the basin would have performed just as efficiently had the flow been introduced in a rectangular cross section, the above conclusion is valid.

BASIN DESIGN

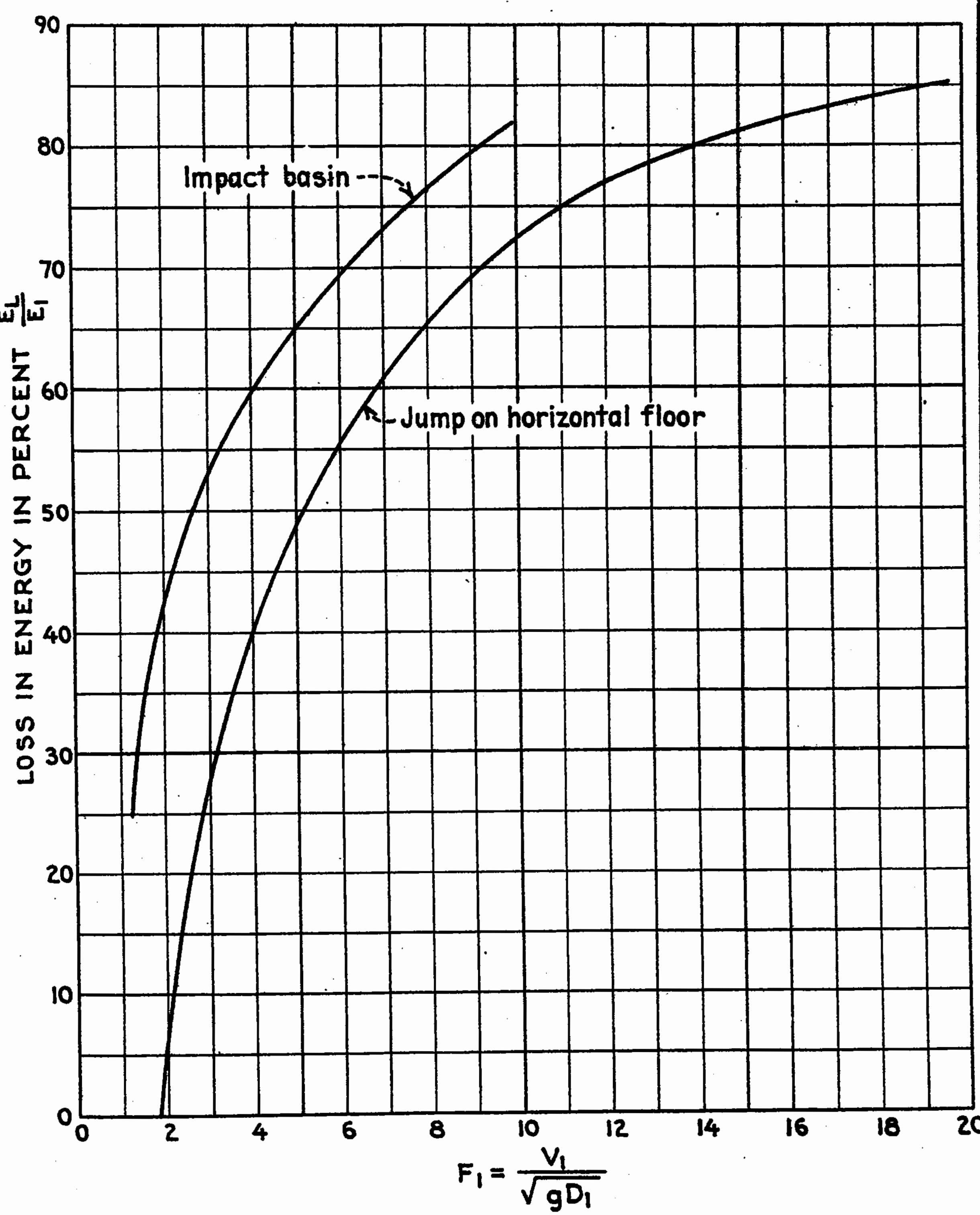
Table 11 and the key drawing, Figure 42, may be used to obtain dimensions for the usual structure operating within usual ranges. However, a further understanding of the design limitations may help the designer to modify these dimensions when necessary for special operating conditions.

The basin dimensions, Columns 4 to 13, are a function of the maximum discharge to be expected, Column 3. Velocity at the stilling basin entrance need not be considered except that it should not exceed about 30 feet per second.

Columns 1 and 2 give the pipe sizes used in designs originating in the Commissioner's Office, Denver, Colorado. These may be changed as necessary, however. These suggested sizes were obtained by assuming the velocity of flow to be 12 feet per second. The pipes shown would then flow full at maximum discharge or they would flow half full at 24 feet per second. The basin operates as well whether a small pipe flowing full or a larger pipe flowing partially full is used. The pipe size may therefore be modified to fit existing conditions, but the relation between structure size and discharge should be maintained as given in the table. In fact, a pipe need not be used at all; an open channel having a width less than the basin width will perform equally as well.

The invert of the entrance pipe, or open channel, should be held at the elevation shown on the drawing of Figure 42, in line with the bottom of the baffle and the top of the end sill, regardless of the size of the pipe selected. The entrance pipe may be tilted downward somewhat without affecting performance adversely. A limit of 15° is a suggested maximum although the loss in efficiency at 20° may not cause excessive erosion. For greater slopes use a horizontal or sloping pipe (up to 15°) 2 or more diameters long just upstream from the stilling basin.

Under certain conditions of flow a hydraulic jump may be expected to form in the downstream end of the pipe sealing the exit end. If the upper end of the pipe is also sealed by incoming flow, a vent may be necessary to prevent pressure fluctuation in the system. A vent to the atmosphere, say one-sixth the pipe diameter, should be installed upstream from the jump.



COMPARISON OF ENERGY LOSSES
IMPACT BASIN AND HYDRAULIC JUMP

The notches shown in the baffle are provided to aid in cleaning out the basin after prolonged nonuse of the structure. When the basin has silted level full of sediment before the start of the spill, the notches provide concentrated jets of water to clean the basin. The basin is designed, however, to carry the full discharge, shown in Table 11, over the top of the baffle if for any reason the space beneath the baffle becomes clogged, Figure 45C. Performance is not as good, naturally, but acceptable. With the basin operating normally, the notches provide some concentration of flow passing over the end sill, resulting in some tendency to scour, Figure 45A. Riprap as shown on the drawing will provide ample protection in the usual installation, but if the best possible performance is desired, it is recommended that the alternate end sill and 45° end walls be used, Figure 45B. The extra sill length reduces flow concentration, scour tendencies, and the height of waves in the downstream channel.

CONCLUSIONS AND RECOMMENDATIONS

The following procedures and rules pertain to the design of Basin VI:

1. Use of Basin VI is limited to cases where the velocity at the entrance to the stilling basin is about 30 feet per second or less.

2. From the maximum expected discharge, determine the stilling basin dimensions, using Table 11, Columns 3 to 13. The use of multiple units side by side may prove economical in some cases.

3. Compute the necessary pipe area from the velocity and discharge. The values in Table 11, Columns 1 and 2, are suggested sizes based on a velocity of 12 feet per second and the desire that the pipe run full at the discharge given in Column 3. Regardless of the pipe size chosen, maintain the relation between discharge and basin size given in the table. An open channel entrance may be used in place of a pipe. The approach channel should be narrower than the basin with invert elevation the same as the pipe.

4. Although tail water is not necessary for successful operation, a moderate depth of tail water will improve the performance. For best performance set the basin so that maximum tail water does not exceed $d + \frac{g}{2}$, Figure 42.

5. The thickness of various parts of the basin as used in the Commissioner's Office, Denver, Colorado, is given in Columns 14 to 18, Table 11.

A. Erosion of channel bed-standard wall and end sill.

B. Less erosion occurs with alternate end sill and wall design.

C. Flow appearance when entire maximum discharge passes over top of baffle during emergency operation.

Channel Erosion and Emergency Operation for Maximum Tabular Discharge
No Tailwater
Impact Type Energy Dissipator - Basin VI

6. The entrance pipe or channel may be tilted downward about 15° without affecting performance adversely. For greater slopes use a horizontal or sloping pipe (up to 15°) 2 or more diameters long just upstream from the stilling basin. Maintain proper elevation of invert at entrance as shown on the drawing.

7. If a hydraulic jump is expected to form in the downstream end of the pipe and the pipe entrance is sealed by incoming flow, install a vent about one-sixth the pipe diameter at any convenient location upstream from the jump.

8. For best possible operation of basin use alternate end sill and 45° wall design shown on Figure 42. Erosion tendencies will be reduced as shown on Figure 45.

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