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ACCURATE water measurement is essential for proper management of this valuable natural resource. There is increasing need for adequate measurement and control as the water demands of our country increase. Hydrologists, engineers, irrigation and power companies as well as many other agencies are interested in accurate measurement of water.

There are many devices to measure water flow. The quantity of water passing a given point can be expressed as the product of the velocity of flow and the cross-sectional area of flow. Most measurement devices take advantage of this simple fact. The weir, orifice, Parshall and type-H flumes are all devices which restrict the flow to a known cross section and, since the velocity is a function of the flow depth it is possible to determine discharge through them. Another popular means of measuring discharge is to determine the velocity with a current meter through any known cross-section. This method is especially adaptable to large stream flow. There are many other devices for measuring water in closed conduits. Since this publication deals with open channel flow they will not be referred to here.

The weir is the oldest and most common device for measuring water discharge in small open channels. James Richno Francis first presented his now classical weir formulas in 1852. (2). This device consists of a bulkhead placed across the channel and provided with a notch through which the water pours. One might consider the weir as an orifice flowing part full. If the channel is rectangular and the bulkhead has a sharp crest extending the full width of the channel the weir has no end contractions and is called a suppressed weir. If the crest width is less than the channel width, the weir has side or end contractions and is referred to as a contracted weir. Most weirs are of the latter type.

Weirs provide an accurate means of measuring water if certain standard conditions are met (3). One of these is that the velocity of approach to the weir should be less than ½ feet per second. In order to meet this condition a relatively large weir pond must be provided ahead of the weir so that the velocity of approach will be negligible. Since the velocity is low in a weir pond it is an ideal location for sediments to be deposited. The velocity of approach increases as silt is deposited and standard weir formulas may no longer be accurate.

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Formulation has been developed for determining discharge when the velocity of approach is greater than ½ fps. (4). However, the procedure is laborious. Several research workers have demonstrated that, when a flat rod is held vertically in a flowing stream, the height of the pile-up on the upstream face of the rod is directly related to the velocity of approach. Wilm and Storey (5) reported on such a rod in 1944. This rod was triangular in cross section and was first held with the apex or sharp edge upstream, then turned to present the flat edge upstream. The difference between the water height on the apex and the flat side of the rod was related to velocity. They found good correlation between rod reading and velocity for velocities between 1 and 8 feet per second. Much earlier in 1921, Steward (6) developed a rod to measure flow over a weir. His rod operated much the same as that described with the Wilm and Storey rod. Much of his work is obscure and details on development and accurary are lacking. The Clausen Weir Rule Company (1) has developed a rule which is held on the weir crest. The flow rate for a given weir width is obtained directly from a scale on the rule.

It was the purpose of this research to determine the relationships between readings taken with a rod held on a weir crest and the discharge flowing over the weir. If a reliable relationship exists then many weirs which have become silted-in can again be used as measuring devices without reconstructing adequate weir ponds. There are many weirs in Idaho, especially in the mountainous areas, which have become inoperative because of silt.

Methods and Materials

The problem involves flow of water over a bulkhead placed in an open channel. The complete study required analysis of the flow resulting from a great many variable conditions. For this reason the work was performed in the irrigation laboratory where variables could be accurately controlled.

THEORY

A weir operating under ideal conditions must have a negligible approach velocity. Also the height of the weir crest above the upstream channel floor must be at least twice the maximum depth of flow over the weir (3). It is logical to assume that neither of these conditions will be met if the weir pond becomes silt laden. In fact, the crest height is an indication of the amount of silting that has taken place and the approach velocity is somewhat dependent upon it. Therefore, the crest height was given consideration in the experiment.

Water flowing over a weir, with a gage held on the crest, is illustrated in Figure 1. To examine this flow experimentally the sup-

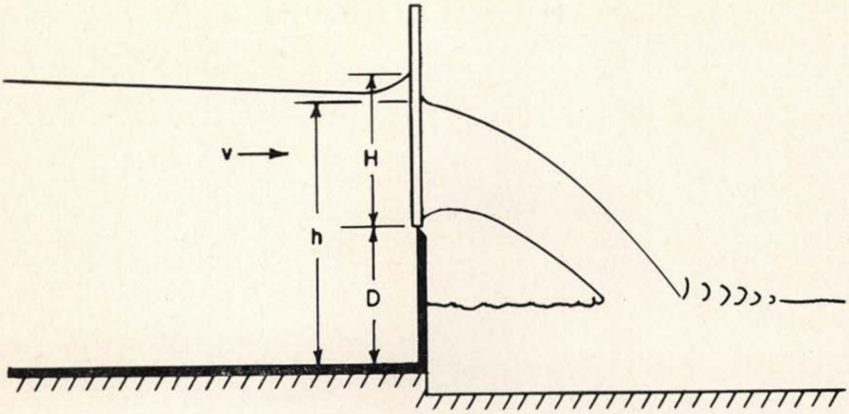

Figure 1.—Gravity flow over a sharp crested weir with a gage held on the crest.

pressed weir was treated separately from the weir with end contractions because of the additional variable, weir length, which was considered with the contracted weir.

SUPPRESSED WEIR: The important variables that describe the flow over a suppressed silted-in weir are q, v, H, D, and g.

Where q = discharge per unit length of weir in ft^2/sec.

v = approach velocity in ft/sec.

H = depth of flow over weir crest as measured with a gage held on the crest in ft.

D = crest height above upstream channel floor in ft.

g = acceleration due to gravity in $ft/sec.^2$

There are other variables such as viscosity, surface tension, and density which might affect the flow. However, in this type of open channel flow their effect can justifiably be assumed negligible.

The assumption was made that the amount of pile-up on the gage, (H+D-h) Figure 1, is a function of the velocity. The velocity, then was eliminated as a variable leaving q, H, D, and g to be studied. One possible combination of dimensionless groups or parameters for these four variables is:

$$q/\sqrt{gD^3} \text{ and } H/D$$

The relationship between these two parameters can be obtained experimentally by varying them so that a range of values for each is obtained and presenting the results graphically.

Of course, the shape and dimensions of the weir rod will affect the value of H. However, for a given shape and dimension the above analysis is valid.

CONTRACTED WEIR: The Contracted weir differs from the suppressed weir in that there is nonuniform flow over the crest length because of the end contractions. The contractions effectively shorten the crest length. This effect varies with the number of contractions and the depth of flow past the weir. Francis (2) proposed correcting for this condition in the following way:

Let $L' = L - 0.10n\ H$

Where L' = effective crest length.

L = measured crest length.

n = number of contractions, usually two.

H = depth of flow over weir crest.

The variables describing the flow over the contracted weir are the same as those over the suppressed weir with the addition of the effective length. This factor was, therefore, introduced in the analysis. Discharge per unit length of weir crest is meaningless since it is not uniform over the crest. Total discharge over the weir must, therefore, be reported. The dimensionless grouping of the variables becomes:

$Q/L'\sqrt{gD^3}$ and H/D

Where Q = total discharge over the weir in ft^3/sec.

L' = effective weir length in ft $= L - 0.2H$

PROCEDURE

To determine relationships between the variables affecting flow, the dimensionless parameters developed in the theory varied through a reasonable range of values. This was accomplished by varying the controllable factors which combine to make up the dimensionless number.

With the suppressed weir $q/\sqrt{gD^3}$ was varied from 0.04 to 35.00 while H/D varied from 0.20 to 15.00. These were obtained by:

Varying D from 0.02 ft to 0.35 ft.

Varying q from 0.05 cfs/ft to 0.35 cfs/ft.

Varying H from 0.065 ft to 0.220 ft.

With the contracted weir $Q/L'\sqrt{gD^3}$ was varied from 0.05 to

40.00 and H/D from 0.20 to 15.00. These variations were obtained by:

Varying D from 0.02 ft to 0.40 ft.

Varying Q from 0.08 cfs to 0.36 cfs.

Varying H from 0.13 ft to 0.32 ft.

A range of approach velocities from 1 to 3 feet per second resulted from the many combinations of tests made.

A special tilting flume was constructed for this study. It was constructed of marine plywood and was 7 feet long, 2 feet wide and 1 foot deep. The tilting feature of the flume allowed a variation of approach velocities for any given discharge.

Weir plates were installed in the end of the flume. The plates were constructed of brass and cut to provide the above mentioned variations in crest height, D. The suppressed weir, of course, extended over the complete flume width of 2 feet. The contracted weir was 8 inches long leaving 8 inch contractions on either side of the weir. Figures 2 and 3 show the flume and weirs in operation.

Figure 2.—Gage reading being taken on suppressed weir.

The flume was installed over the existing large laboratory flume in order to utilize the recirculating system. Water was pumped from the sump beneath the laboratory floor, through the test flume,

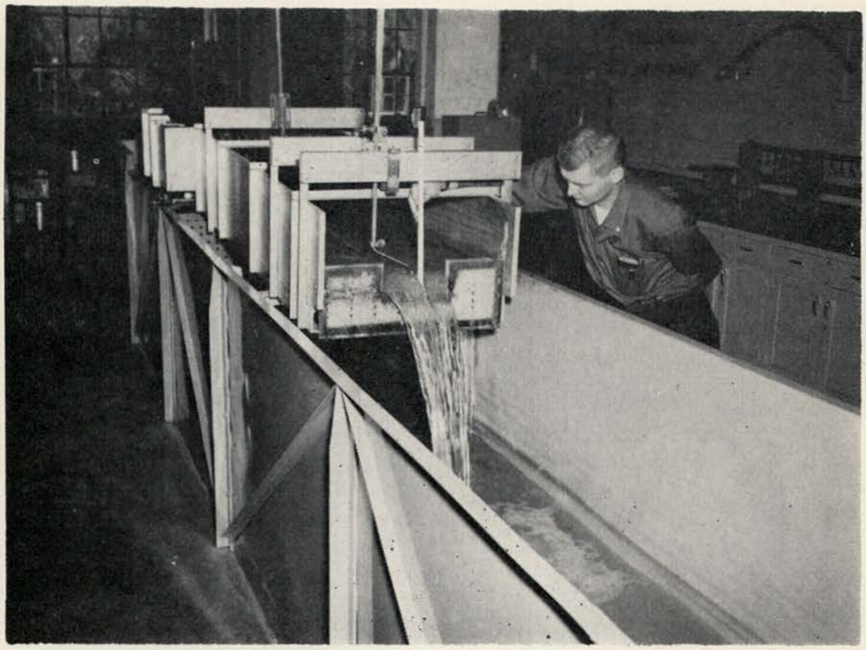

Figure 3.—Gage reading being taken on contracted weir.

into the permanent flume and hence into the sump. Discharge was controlled by a gate valve between the pump and the flume entrance. It was accurately measured through a calibrated water meter.

The important measurement considered was the depth of flow over the weir as measured with a gage held on the weir crest. The gage used was constructed of wood and ½ inch square in cross section. A scale was placed on one face of the gage which allowed readings to the nearest 0.01 feet. In operation, the gage was held vertically on the crest and a reading made on the upstream face of the gage at the point of maximum water rise. Of course, this reading (H) was greater than the true depth because it consisted of the pile-up due to velocity as well as the true depth. The actual depth of flow over the weir was measured with a point gage placed directly over the weir crest. Figures 2 and 3 illustrate the method of reading the gage height. The point gage at the weir can also be seen.

DATA COLLECTION

One complete trial consisted of operating the apparatus with a given weir in place and with a given discharge over the weir while the flume was at a given slope. Water was turned into the flume and the flow rate was adjusted to the desired amount with the gate valve. After the flow had become stabilized; discharge, Q, and gage height, H, measurements were taken. At each flow condition, five readings were taken on these items. The recorded value was the average of these readings. The value of q (discharge per foot of weir length) for the suppressed weir was calculated by dividing total discharge, Q, by the weir length, L.

Data collected with the suppressed weir included operating the flume at 4 different slopes, using 6 separate discharge rates and 5 weir heights. This involved 120 separate tests with this weir. When these data were plotted it was found that there was a great deal of duplication. The number of trials with the contracted weir was therefore reduced to 3 weir heights on 3 slopes with 4 discharge rates making a total of 36 individual tests.

Experimental Results

The dimensionless grouping of variables, discussed in the theory, were calculated from the data. Each particular test-run produced one set of dimensionless numbers. After all tests had been calculated the results were plotted. Figure 4 shows this information for the suppressed weir while Figure 5 represents the contracted weir.

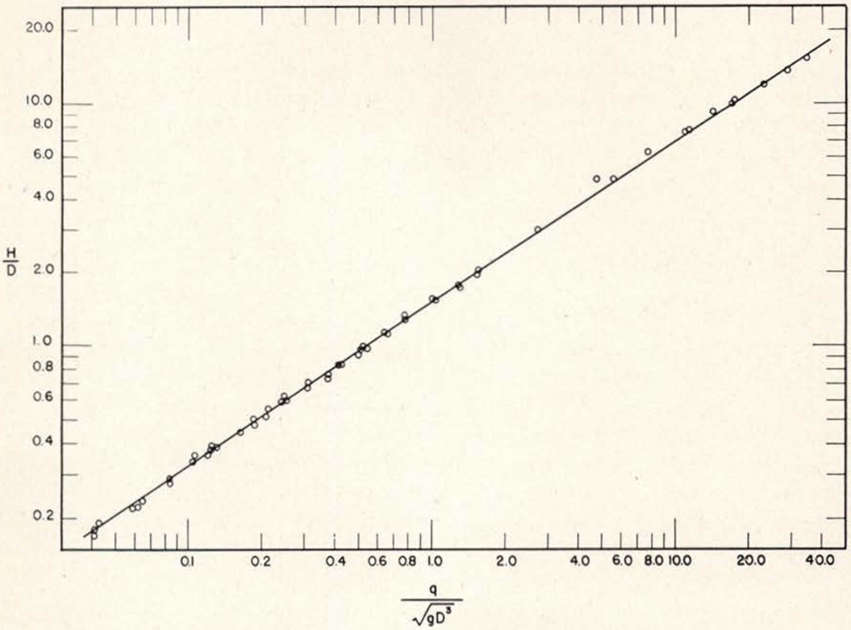

Figure 4.—Relationship of dimensionless grouping of variables for suppressed weir flow.

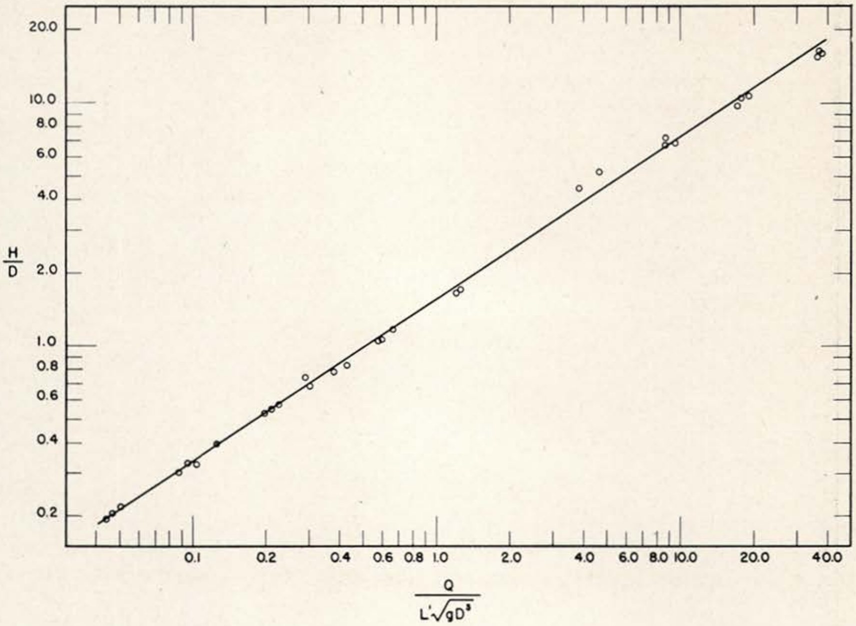

Figure 5.—Relationship of dimensionless grouping of variables for contracted weir flow.

These plots demonstrate the extremely close relationship between the parameters. In fact, the data plotted in a straight line on logrithmic paper indicating an exponential relationship between the variables. It was therefore possible to develop equations relating discharge to the other variables. The resulting equations were as follows:

Suppressed weir $\quad q = 3.310\ D^{-0.009}\ H^{1.549} \quad (1)$

Contracted weir $\quad Q = 3.584\ (L - 0.2H)\ D^{0.009}\ H^{1.597} \quad (2)$

These equations were found by the multiple regression method rather than the graphicial method because of the inaccuracies involved in the latter. The value of the weir crest height, D, above the channel floor has a very small effect on the discharge as measured with the gage because, regardless of the value of D, when it is taken to such a small power the result approaches 1. Neglecting D in the equations would result in an error of less than two percent. The accuracy of any operating weir is undoubtedly no better than this. Omitting D as a variable in the analysis was therefore justified.

This was done and a direct relationship between discharge and depth of flow was obtained. These relationships appear in Figures 6 and 7 and the equations resulting from multiple regression were as follows:

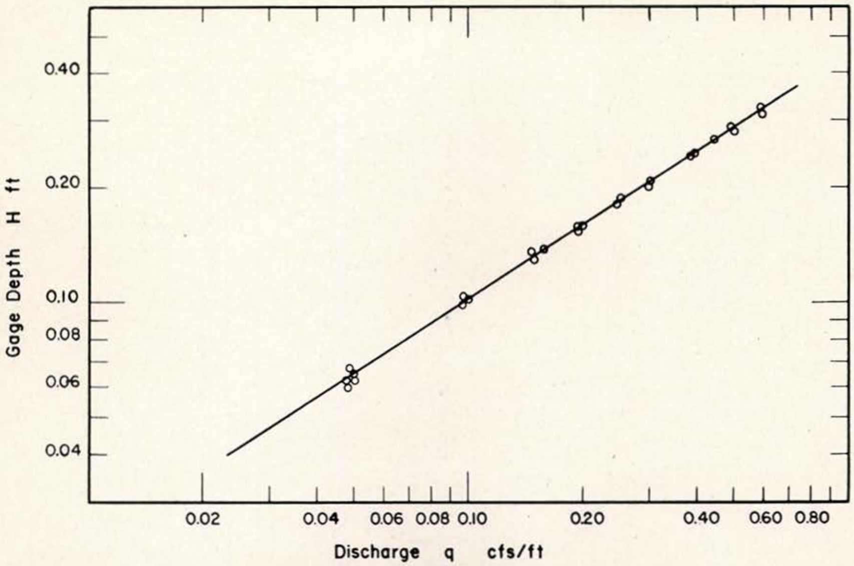

Figure 6.—Gage depth-discharge relationship for flow over a suppressed weir.

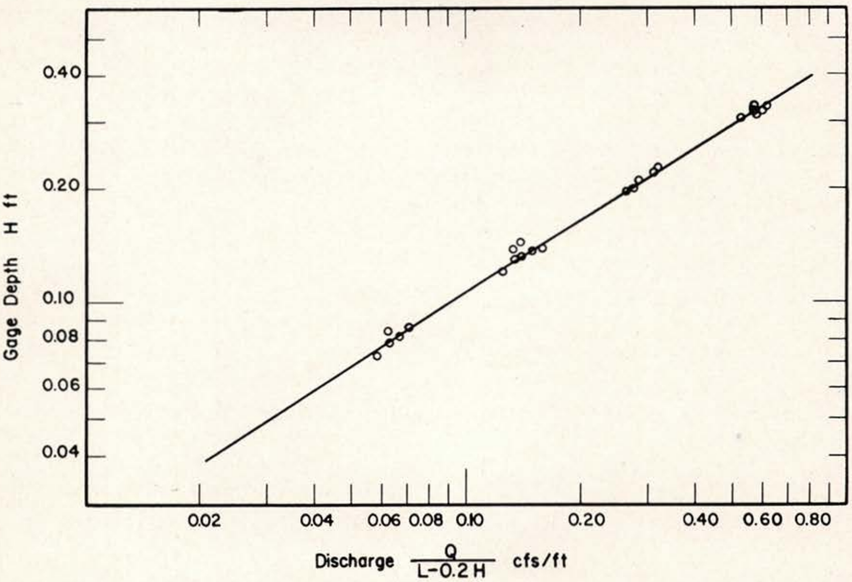

Figure 7.—Gage depth-discharge relationship for flow over a contracted weir.

Suppressed weir $q = 3.369\ H^{1.549}$ (3)

Contracted weir $Q = 3.496\ (L - 0.2H)\ H^{1.593}$ (4)

In order to determine the accuracy obtainable with these four equations a correlation was made between calculated discharge and measured discharge. The resulting correlation coefficients were:

Weir Type	Equation	Coefficient
Suppressed	1	0.999
Suppressed	3	0.988
Contracted	2	0.998
Contracted	4	0.990

It is obvious that very little is gained by using equations 1 and 2 when 3 and 4 are almost as accurate. Equations 3 and 4, then, represent discharge over a weir which has an appreciable approach velocity. The discharge (q) in equation 3 represents the discharge per foot of weir length while the discharge (Q) in equation 4 represents the total discharge over a weir of length L. The value of the depth of flow H was measured with a gage held on the weir crest. This depth was measured with a gage which was square in cross section being ½ inch by ½ inch. These formulas are not valid if a gage of any other size or configuration is used.

Tables 1 and 2 have been calculated and are presented in order that this method of measurement can be used without laboriously using the formula for each specific flow. Information on four weir widths is presented. It will be noticed that values of H greater than one-third the crest length are not given. It has been found that for greater depths than this the contractions are not complete and measurments are inaccurate unless special precautions are taken. Since this study did not involve measurements at these depths it is doubtful that these formulas apply.

Summary

Weirs that have silted-in no longer operate as standard weirs. The velocity of approach becomes greater than ½ fps and is greater than standard conditions. The weir crest height above the channel bottom is reduced. For these reasons the usual method of measurement over them is inaccurate.

This study has demonstrated that it is possible to utilize these weirs as accurate measuring devices if the depth of flow is measured with a gage held on the weir crest. The value of the depth measured in this manner includes the true depth of flow and a certain amount of pile-up on the gage face caused by the approach-velocity.

The study related the depth of flow measured with a ½ inch square gage to the discharge over the weir. It was found that for the method of measurement used, the crest height above the channel floor has very little effect on the measurement and may be eliminated as a variable.

Equations were developed relating discharge to gage depth and tables which may be used to determine discharge are presented.

Table 1.—Discharge over suppressed rectangular weirs in cubic feet per second. Depth measured with ½ inch square gage held on the weir crest.*

Gage Depth (ft)	Crest Length of Weir (ft) 1.0	1.5	2.0	3.0	Gage Depth (ft)	Crest Length of Weir (ft) 1.0	1.5	2.0	3.0
0.05	0.033	0.050	0.067	0.100	0.55	....	....	2.670	4.005
0.06	0.043	0.065	0.086	0.129	0.56	....	....	2.746	4.119
0.07	0.055	0.082	0.110	0.164	0.57	....	....	2.822	4.233
0.08	0.067	0.101	0.134	0.202	0.58	....	....	2.898	4.347
0.09	0.081	0.121	0.162	0.242	0.59	....	....	2.976	4.464
0.10	0.095	0.143	0.190	0.286	0.60	....	....	3.054	4.581
0.11	0.110	0.165	0.221	0.331	0.61	....	....	3.134	4.701
0.12	0.126	0.190	0.253	0.379	0.62	....	....	3.213	4.820
0.13	0.143	0.214	0.285	0.428	0.63	....	....	3.294	4.941
0.14	0.160	0.240	0.320	0.481	0.64	....	....	3.374	5.061
0.15	0.178	0.268	0.357	0.535	0.65	....	....	3.457	5.186
0.16	0.197	0.296	0.394	0.591	0.66	....	....	3.540	5.310
0.17	0.217	0.325	0.433	0.650	0.67	....	....	3.623	5.435
0.18	0.237	0.355	0.473	0.710	0.68	....	....	3.708	5.561
0.19	0.257	0.385	0.514	0.770	0.69	....	....	....	5.689
0.20	0.278	0.417	0.557	0.835	0.70	....	....	....	5.817
0.21	0.300	0.450	0.599	0.899	0.71	....	....	....	5.946
0.22	0.323	0.486	0.646	0.968	0.72	....	....	....	6.076
0.23	0.346	0.518	0.691	1.037	0.73	....	....	....	6.207
0.24	0.369	0.554	0.738	1.107	0.74	....	....	....	6.339
0.25	0.393	0.590	0.786	1.179	0.75	....	....	....	6.473
0.26	0.418	0.628	0.837	1.255	0.76	....	....	....	6.607
0.27	0.443	0.665	0.886	1.329	0.77	....	....	....	6.741
0.28	0.469	0.701	0.938	1.407	0.78	....	....	....	6.878
0.29	0.495	0.743	0.990	1.485	0.79	....	....	....	7.015
0.30	0.522	0.783	1.044	1.566	0.80	....	....	....	7.153

0.31	0.549	0.824	1.098	1.647	0.81	....	....	....	7.292
0.32	0.576	0.864	1.152	1.728	0.82	....	....	....	7.432
0.33	0.605	0.908	1.210	1.815	0.83	....	....	....	7.573
0.34	0.633	0.950	1.266	1.899	0.84	....	....	....	7.713
0.35	....	0.995	1.326	1.989	0.85	....	....	....	7.858
0.36	....	1.038	1.384	2.076	0.86	....	....	....	8.009
0.37	....	1.083	1.444	2.166	0.87	....	....	....	8.146
0.38	....	1.130	1.506	2.259	0.88	....	....	....	8.291
0.39	....	1.176	1.568	2.352	0.89	....	....	....	8.438
0.40	....	1.223	1.630	2.445	0.90	....	....	....	8.584
0.41	....	1.269	1.692	2.538	0.91	....	....	....	8.733
0.42	....	1.319	1.758	2.637	0.92	....	....	....	8.882
0.43	....	1.367	1.822	2.733	0.93	....	....	....	9.022
0.44	....	1.418	1.890	2.835	0.94	....	....	....	9.183
0.45	....	1.467	1.956	2.934	0.95	....	....	....	9.335
0.46	....	1.5[illegible]8	2.024	3.036	0.96	....	....	....	9.487
0.47	....	1.568	2.090	3.135	0.97	....	....	....	9.641
0.48	....	1.620	2.160	3.240	0.98	....	....	....	9.796
0.49	....	1.673	2.230	3.345	0.99	....	....	....	9.951
0.50	....	1.727	2.303	3.454	1.00	....	....	....	10.107
0.51	....	....	2.375	3.562					
0.52	....	....	2.446	3.669					
0.53	....	....	2.519	3.779					
0.54	....	....	2.594	3.891					

*Computed with the formula $Q = 3.369\ L\ H^{1.549}$

Table 2.—Discharge over contracted rectangular weirs in cubic feet per second. Depth measured with ½ inch square gage held on the weir crest.*

Gage Depth (ft)	Crest Length of Weir (ft) 1.0	1.5	2.0	3.0	Gage Depth (ft)	Crest Length of Weir (ft) 1.0	1.5	2.0	3.0
0.05	0.029	0.044	0.059	0.088	0.55	....	....	2.549	3.898
0.06	0.039	0.059	0.079	0.118	0.56	....	....	2.621	4.009
0.07	0.050	0.075	0.100	0.151	0.57	....	....	2.693	4.121
0.08	0.062	0.093	0.124	0.187	0.58	....	....	2.754	4.217
0.09	0.074	0.112	0.150	0.225	0.59	....	....	2.839	4.347
0.10	0.087	0.132	0.177	0.266	0.60	....	....	2.913	4.462
0.11	0.102	0.154	0.205	0.309	0.61	....	....	2.987	4.578
0.12	0.116	0.176	0.236	0.355	0.62	....	....	3.063	4.695
0.13	0.132	0.200	0.268	0.403	0.63	....	....	3.138	4.813
0.14	0.148	0.225	0.301	0.453	0.64	....	....	3.215	4.932
0.15	0.165	0.250	0.335	0.506	0.65	....	....	3.291	5.052
0.16	0.183	0.277	0.371	0.560	0.66	....	....	3.369	5.172
0.17	0.201	0.305	0.409	0.616	0.67	....	....	3.447	5.294
0.18	0.219	0.333	0.447	0.675	0.68	....	....	3.525	4.417
0.19	0.239	0.363	0.487	0.735	0.69	....	....	....	5.540
0.20	0.258	0.393	0.528	0.797	0.70	....	....	....	5.665
0.21	0.279	0.424	0.570	0.861	0.71	....	....	....	5.790
0.22	0.300	0.456	0.613	0.926	0.72	....	....	....	5.916
0.23	0.321	0.489	0.657	0.994	0.73	....	....	....	6.044
0.24	0.343	0.523	0.703	1.062	0.74	....	....	....	6.172
0.25	0.365	0.557	0.749	1.133	0.75	....	....	....	6.301
0.26	0.388	0.592	0.796	1.205	0.76	....	....	....	6.441
0.27	0.411	0.628	0.845	1.279	0.77	....	....	....	6.561
0.28	0.434	0.664	0.895	1.355	0.78	....	....	....	6.693
0.29	0.458	0.702	0.945	1.432	0.79	....	....	....	6.825
0.30	0.483	0.740	0.996	1.510	0.80	....	....	....	6.958

0.31	0.508	0.778	1.049	1.590	0.81	....	....	....	7.092
0.32	0.533	0.817	1.102	1.671	0.82	....	....	....	7.227
0.33	0.558	0.857	1.156	1.754	0.83	....	....	....	7.363
0.34	0.584	0.898	1.211	1.838	0.84	....	....	....	7.500
0.35	....	0.939	1.267	1.924	0.85	....	....	....	7.637
0.36	....	0.981	1.324	2.011	0.86	....	....	....	7.775
0.37	....	1.023	1.382	2.099	0.87	....	....	....	7.914
0.38	....	1.066	1.440	2.188	0.88	....	....	....	8.054
0.39	....	1.109	1.499	2.279	0.89	....	....	....	8.194
0.40	....	1.153	1.559	2.372	0.90	....	....	....	8.335
0.41	....	1.198	1.620	2.465	0.91	....	....	....	8.477
0.42	....	1.243	1.682	2.560	0.92	....	....	....	8.620
0.43	....	1.289	1.744	2.656	0.93	....	....	....	8.764
0.44	....	1.335	1.807	2.753	0.94	....	....	....	8.908
0.45	....	1.382	1.871	2.851	0.95	....	....	....	9.053
0.46	....	1.429	1.936	2.951	0.96	....	....	....	9.199
0.47	....	1.476	2.001	3.051	0.97	....	....	....	9.345
0.48	....	1.525	2.067	3.153	0.98	....	....	....	9.492
0.49	....	1.573	2.134	3.256	0.99	....	....	....	9.640
0.50	....	1.622	2.202	3.361	1.00	....	....	....	9.789
0.51	....	....	2.270	3.466					
0.52	....	....	2.339	3.572					
0.53	....	....	2.408	3.680					
0.54	....	....	2.479	3.789					

*Computed with the formula $Q = 3.496\,(L - 0.2H)\,H^{1.593}$

References

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