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RBC BROAD-CRESTED WEIRS FOR CIRCULAR SEWERS AND PIPES

A.J. CLEMMENS[1], M.G. BOS[2] and J.A. REPLOGLE[1]

[1] *United States Water Conservation Laboratory, Agricultural Research Service, U.S. Department of Agriculture, Phoenix, AZ 85040 (U.S.A.)*
[2] *International Institute for Land Reclamation and Improvement (I.L.R.I.), Wageningen (The Netherlands)*

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ABSTRACT

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In recent years, significant advancements have been made in the practical application of long-throated flumes and broad-crested weirs for flow measurements in irrigation canals. The modified RBC (Replogle—Bos—Clemmens) broad-crested weir has many advantages over related open-channel flow devices. These include high accuracy and reliability for a wide variety of shapes, low head-loss requirements which are predictable, and relatively simple inexpensive construction. In this paper we have extended the application of these weirs to circular pipes flowing partially full. The theoretical equations are presented for ideal flow from which approximate ratings can be obtained to within a reasonable accuracy with an empirical discharge coefficient. However, a mathematical model is available which accurately predicts these ratings by directly accounting for the effects of friction. The ratings for a wide variety of shapes and sizes of these weirs were computed with the model and fit to an empirical equation. The constants for this equation are plotted graphically for easy use. The resulting ratings should be well within ±3%. Design examples are given which show how to select the flume dimensions for maintaining free-flowing conditions (modular flow) and for minimizing sediment deposition. Once constructed, the rating for a given flume can be determined even when not constructed as planned.

INTRODUCTION

Many methods and refinements have been reported for measuring the flow rate in or from circular sewers and other circular pipes (Palmer and Bowlus, 1936; Singer, 1936; King, 1954; Wells and Gotaas, 1958; Diskin, 1963a, b, 1976; Bos, 1976). Most of the developed measurement devices were not widely accepted because they were one-of-a-kind laboratory tested constructions with limited ranges of application and limited design flexibility.

Another group of structures, the circular and U-shaped long-throated flumes, is of the same hydraulic family as the weirs of this paper and can be adapted to a wide variety of conditions (Replogle, 1975; Bos, 1976, 1977a). The normal way to construct these flumes is to insert a half or full section of smaller-diameter pipe into the original pipe to form the throat section and then plaster a ramp in place. This procedure is cumbersome, costly, and may yield a flume with a discharge capacity which is considerably less than that of the sewer or pipe. To overcome or minimize these limitations and to simplify the construction, the RBC weir was developed. This modification of the broad-crested weir can be treated by existing theory so that rating tables can be produced for each size of weir and the minimum required head loss over the weir can be calculated.

As shown in Fig. 1, the structure is reduced to a truly horizontal weir sill with length, L, and a ramp which may slope between 2:1 (two units horizontal to one unit vertical) and 3:1. As with the related RBC weirs for trapezoidal canals, the horizontal crest must be finished carefully to the required width, the ramp(s) can be hand-plastered while the rest of the pipe provides the remaining surfaces of the structure. Thus construction is limited to one accurately finished surface.

The purpose of this paper is: (1) to present the general theory of flow over modified broad-crested weirs; (2) to present head—discharge relationships for broad-crested weirs installed in circular channels; and (3) to present design methods for minimizing the effects of these weirs on the flow of water and movement of sediment in the pipe.

THEORY

The dimensions and hydraulic properties of broad-crested weirs are very similar to those described by Wells and Gotaas (1958). These were improved upon by Akers and Harrison (1963) and further refined by Replogle (1975) and extended by Bos (1976, 1977a) and Bos and Reinink (1981). The general flow profile over broad-crested weirs and the symbols for describing the flow and the weir dimensions are given in Fig. 1.

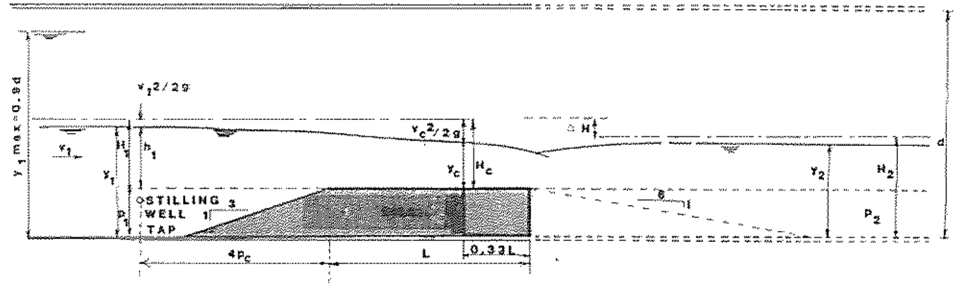
From the above references, it is known that for a broad-crested weir with critical flow at the control section the following equations are valid:

$$H_1 = \alpha_1 v_1^2/2g + h_1; \qquad H_c = \alpha_c v_c^2/2g + y_c \qquad (1), (2)$$

$$Q = C_d A_c [2g(H_1 - y_c)]^{0.5} \quad \text{and} \quad v_c^2/2g = A_c/2B_c \qquad (3), (4)$$

where the subscripts 1 or c refer to the gauging station and the control section, respectively (see Fig. 1), and H_1 = total energy head at the gauging station (m); H_c = total energy head at the control section (m); α_1 = velocity distribution coefficient (dimensionless); v = average flow velocity (m s^{-1}); g = acceleration due to gravity (m s^{-2}); h_1 = upstream sill-referenced head (m); y_c = critical depth of water at control section (m); Q = rate of flow (m^3 s^{-1}); A_c = wetted area at control section (m^2); B_c = water surface width at control section (m); and C_d = discharge coefficient (dimensionless).

The latter discharge coefficient has a value slightly below 1.0 and corrects for the common assumptions that $\alpha_1 = \alpha_c = 1.0$, that streamlines at both



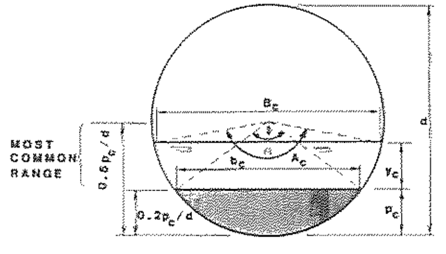


Fig. 1. RBC broad-crested weir in circular channel.

the gauging station and control section are straight and parallel, and that:

$$H_1 = H_c \tag{5}$$

This equation assumes that no energy is lost between the gauging station and control section. For relatively low heads ($H_1/L \leqslant 0.5$), however, this is the primary factor influencing the C_d-value. At higher heads, stream-line curvature at the control section has an increasingly positive influence on the C_d-value. For C_d-values see Fig. 2.

Eq. 3 may also be written in terms of dimensionless ratios for flow in circular channels as described by Bos (1976):

$$Q/d^{2.5}g^{0.5} = C_d(A_c/d^2)[2(H_1/d - y_c/d)]^{0.5} \tag{6}$$

From the geometry, the following relationships can be derived:

$$A_s = d^2(\phi - \sin\phi)/8; \qquad A_t = d^2(\theta - \sin\theta)/8 \qquad (7), (8)$$

$$B_c = d\sin(\tfrac{1}{2}\theta); \qquad b_c = d\sin(\tfrac{1}{2}\phi) \qquad (9), (10)$$

and

$$p_c = d[1 - \cos(\tfrac{1}{2}\phi)]/2 \tag{11}$$

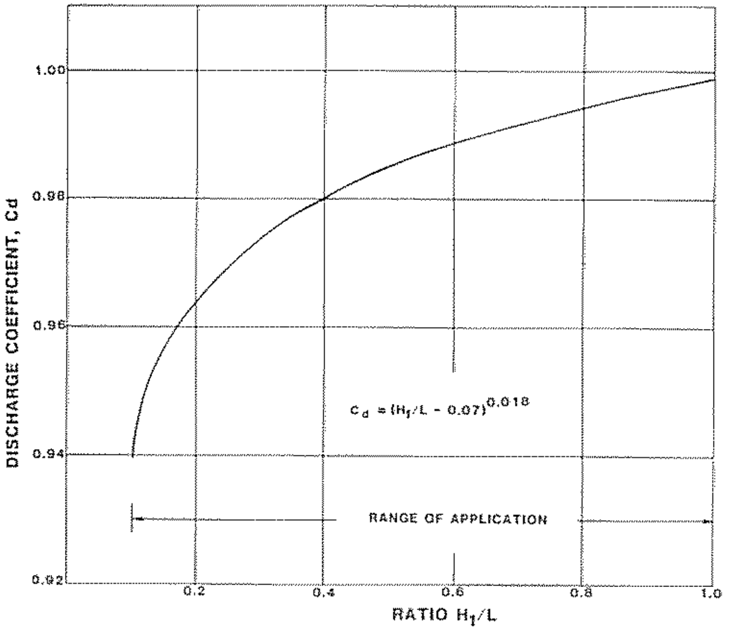


Fig. 2. Values of C_d as a function of H_1/L for broad-crested weirs and long-throated flumes.

where A_s = area of sill; A_t = total area of sill and wetted flow area at the control section; b_c = sill width; and p_c = sill height at the control section (see Fig. 1). (If the pipe is horizontal with no pipe down-sets, note that $p_c = p_1 = p_2$.) The wetted area of the control section, A_c, in eq. 6 is found from:

$$A_c = A_t - A_s = d^2(\theta - \phi + \sin\phi - \sin\theta)/8 \tag{12}$$

Substituting eq. 12 into eq. 4 gives:

$$v_c^2/2g = \tfrac{1}{16}\, d(\theta - \phi + \sin\phi - \sin\theta)/\sin(\tfrac{1}{2}\theta) \tag{13}$$

which in combination with eq. 2 and 5 with $\alpha_c = 1.0$ yields:

$$H_1/d - y_c/d = \tfrac{1}{16}(\theta - \phi + \sin\phi - \sin\theta)/\sin(\tfrac{1}{2}\theta) \tag{14}$$

Substituting eqs. 12 and 14 into eq. 6 results in:

$$Q/d^{2.5}g^{0.5} = \tfrac{1}{8}(\theta - \phi + \sin\phi - \sin\theta)^{1.5}/[8\sin(\tfrac{1}{2}\theta)]^{0.5} \tag{15}$$

where $C_d = 1.0$ for $H_1 = H_c$. Thus for given values of p_c/d (and thus ϕ) and y_c/d (and thus θ), eqs. 14 and 15 can be solved for H_1/d and $Q/d^{2.5}g^{0.5}$. These values were computed and are given in Table I. A similar table for circular control sections is given in Bos (1976, p. 26; 1977a, p. 123). In using Table I, note that the velocity head is included and do not confuse h_1 with H_1.

The values of Q computed from Table I should be adjusted with a C_d-value from Fig. 2.

TABLE I

Ratios for determining the discharge of an RBC broad-crested weir, $C_d = 1.0$, $\alpha_c = 1.0$, $H_1 = H_c$

$(p_c + H_1)/d$	Ratio $Q/d^{2.5} g^{0.5}$ for $p_c/d =$							
	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
0.16	0.0004							
0.17	0.0011							
0.18	0.0021							
0.19	0.0032							
0.20	0.0045							
0.21	0.0060	0.0004						
0.22	0.0076	0.0012						
0.23	0.0094	0.0023						
0.24	0.0113	0.0036						
0.25	0.0133	0.0050						
0.26	0.0155	0.0066	0.0005					
0.27	0.0177	0.0084	0.0013					
0.28	0.0201	0.0103	0.0025					
0.29	0.0226	0.0124	0.0038					
0.30	0.0252	0.0145	0.0054					
0.31	0.0280	0.0169	0.0071	0.0005				
0.32	0.0308	0.0193	0.0090	0.0014				
0.33	0.0337	0.0219	0.0110	0.0026				
0.34	0.0368	0.0245	0.0132	0.0040				
0.35	0.0399	0.0273	0.0155	0.0057				
0.36	0.0432	0.0302	0.0179	0.0075	0.0005			
0.37	0.0465	0.0332	0.0205	0.0094	0.0015			
0.38	0.0500	0.0363	0.0232	0.0115	0.0027			
0.39	0.0535	0.0396	0.0260	0.0138	0.0042			
0.40	0.0571	0.0429	0.0289	0.0162	0.0059			
0.41	0.0609	0.0463	0.0320	0.0187	0.0077	0.0005		
0.42	0.0647	0.0498	0.0351	0.0214	0.0097	0.0015		
0.43	0.0686	0.0534	0.0383	0.0242	0.0119	0.0028		
0.44	0.0726	0.0571	0.0417	0.0271	0.0143	0.0043		
0.45	0.0767	0.0609	0.0451	0.0301	0.0167	0.0060		
0.46	0.0809	0.0648	0.0487	0.0332	0.0193	0.0079	0.0005	
0.47	0.0851	0.0688	0.0523	0.0365	0.0220	0.0100	0.0015	
0.48	0.0895	0.0729	0.0561	0.0398	0.0249	0.0122	0.0028	
0.49	0.0939	0.0770	0.0599	0.0432	0.0279	0.0145	0.0043	
0.50	0.0984	0.0813	0.0638	0.0468	0.0309	0.0170	0.0061	
0.51	0.1030	0.0856	0.0678	0.0504	0.0341	0.0197	0.0080	0.0005
0.52	0.1076	0.0900	0.0719	0.0541	0.0374	0.0224	0.0101	0.0015
0.53	0.1124	0.0945	0.0761	0.0579	0.0408	0.0253	0.0123	0.0028
0.54	0.1172	0.0990	0.0803	0.0618	0.0443	0.0283	0.0147	0.0044
0.55	0.1221	0.1037	0.0847	0.0658	0.0479	0.0314	0.0172	0.0061
0.56	0.1270	0.1084	0.0891	0.0699	0.0515	0.0346	0.0198	0.0080
0.57	0.1320	0.1132	0.0936	0.0741	0.0553	0.0379	0.0226	0.0101

TABLE I (*continued*)

$(p_c + H_t)/d$	Ratio $Q/d^{2.5}g^{0.5}$ for p_c/d =							
	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
0.58	0.1372	0.1180	0.0981	0.0783	0.0592	0.0413	0.0255	0.0123
0.59	0.1423	0.1230	0.1028	0.0826	0.0631	0.0448	0.0285	0.0147
0.60	0.1476	0.1280	0.1075	0.0870	0.0671	0.0484	0.0316	0.0172
0.61		0.1330	0.1123	0.0915	0.0712	0.0521	0.0348	0.0198
0.62		0.1382	0.1172	0.0960	0.0754	0.0559	0.0381	0.0225
0.63		0.1434	0.1221	0.1006	0.0797	0.0597	0.0415	0.0254
0.64		0.1486	0.1271	0.1053	0.0840	0.0637	0.0449	0.0283
0.65		0.1540	0.1321	0.1101	0.0884	0.0677	0.0485	0.0314
0.66		0.1593	0.1373	0.1149	0.0929	0.0718	0.0522	0.0346
0.67		0.1648	0.1424	0.1198	0.0974	0.0759	0.0559	0.0378
0.68		0.1703	0.1477	0.1247	0.1020	0.0802	0.0597	0.0412
0.69		0.1759	0.1530	0.1297	0.1067	0.0845	0.0636	0.0446
0.70		0.1815	0.1584	0.1348	0.1114	0.0888	0.0676	0.0481
0.71		0.1872	0.1638	0.1399	0.1162	0.0933	0.0716	0.0517
0.72		0.1929	0.1692	0.1451	0.1211	0.0978	0.0757	0.0554
0.73		0.1987	0.1748	0.1503	0.1260	0.1023	0.0799	0.0591
0.74		0.2045	0.1804	0.1556	0.1310	0.1070	0.0841	0.0629
0.75		0.2104	0.1860	0.1610	0.1360	0.1116	0.0884	0.0668
0.76		0.2163	0.1917	0.1663	0.1411	0.1164	0.0928	0.0707
0.77		0.2223	0.1974	0.1718	0.1462	0.1211	0.0972	0.0747
0.78		0.2283	0.2031	0.1773	0.1514	0.1260	0.1016	0.0788
0.79		0.2344	0.2090	0.1828	0.1566	0.1309	0.1061	0.0829
0.80		0.2405	0.2148	0.1884	0.1618	0.1358	0.1107	0.0870
0.81		0.2466	0.2207	0.1940	0.1672	0.1408	0.1153	0.0913
0.82		0.2528	0.2267	0.1997	0.1725	0.1458	0.1200	0.0955
0.83		0.2591	0.2326	0.2054	0.1779	0.1508	0.1247	0.0998
0.84		0.2653	0.2386	0.2111	0.1833	0.1559	0.1294	0.1042
0.85		0.2716	0.2447	0.2169	0.1888	0.1611	0.1342	0.1086
0.86		0.2780	0.2508	0.2227	0.1943	0.1662	0.1390	0.1130
0.87		0.2843	0.2569	0.2285	0.1998	0.1714	0.1438	0.1175
0.88		0.2907	0.2630	0.2344	0.2054	0.1767	0.1487	0.1220
0.89		9.2971	0.2692	0.2403	0.2110	0.1819	0.1536	0.1266
0.90		0.3036	0.2754	0.2462	0.2166	0.1872	0.1586	0.1311
0.91		0.3101	0.2816	0.2521	0.2223	0.1926	0.1636	0.1357
0.92		0.3166	0.2879	0.2581	0.2279	0.1979	0.1686	0.1404
0.93		0.3231	0.2942	0.2641	0.2336	0.2033	0.1736	
0.94		0.3297	0.3005	0.2701	0.2394	0.2087		
0.95		0.3362	0.3068	0.2762	0.2451			
0.96		0.3428	0.3131	0.2823	0.2509			
0.97		0.3494	0.3195	0.2883				
0.98		0.3561	0.3259	0.2944				
0.99		0.3627	0.3323					
1.00		0.3694	0.3387					
1.01		0.3760	0.3451					
1.02		0.3827						
1.03		0.3894						
1.04		0.3961						

TABLE II

Relative area and top width for flow in circular channels

p_c/d or y_1/d	A_s/d or A_1/d	p_c/d or B_1/d	p_c/d or y_1/d	A_s/d or A_1/d	p_c/d or B_1/d
0.01	0.0013	0.1990	0.26	0.1623	0.8773
0.02	0.0037	0.2800	0.27	0.1711	0.8879
0.03	0.0069	0.3412	0.28	0.1800	0.8980
0.04	0.0105	0.3919	0.29	0.1890	0.9075
0.05	0.0147	0.4359	0.30	0.1982	0.9165
0.06	0.0192	0.4750	0.31	0.2074	0.9250
0.07	0.0242	0.5103	0.32	0.2167	0.9330
0.08	0.0294	0.5426	0.33	0.2260	0.9404
0.09	0.0350	0.5724	0.34	0.2355	0.9474
0.10	0.0409	0.6000	0.35	0.2450	0.9539
0.11	0.0470	0.6258	0.36	0.2546	0.9600
0.12	0.0534	0.6499	0.37	0.2642	0.9656
0.13	0.0600	0.6726	0.38	0.2739	0.9708
0.14	0.0688	0.6940	0.39	0.2836	0.9755
0.15	0.0739	0.7141	0.40	0.2934	0.9798
0.16	0.0811	0.7332	0.41	0.3032	0.9837
0.17	0.0885	0.7513	0.42	0.3132	0.9871
0.18	0.0961	0.7684	0.43	0.3229	0.9902
0.19	0.1039	0.7846	0.44	0.3328	0.9928
0.20	0.1118	0.8000	0.45	0.3428	0.9950
0.21	0.1199	0.8146	0.46	0.3527	0.9968
0.22	0.1281	0.8285	0.47	0.3627	0.9982
0.23	0.1365	0.8417	0.48	0.3727	0.9992
0.24	0.1449	0.8542	0.49	0.3827	0.9998
0.25	0.1535	0.8660	0.50	0.3927	1.0000

To transfer these H_1-values to h_1-values that will be measured, calculate h_1 from eq. 1 which can be written:

$$h_1 = H_1 - \alpha_1 Q^2/2gA_1^2 \tag{16}$$

The α_1 in eq. 16 is added to maintain the proper h_1 to H_1 relationship. The effect of α_1 on the value of C_d determined through H_1/L is minor. It is also appropriate to use $\alpha_1 = 1.04$ in this case (which assumes a fully developed profile) since Fig. 2 was developed primarily from calibrations where $\alpha_1 = 1.04$ was used. Dimensionless A_1-values can be obtained from Table II. Once a value of h_1 is calculated, A_1 can be recomputed from the Tables I and II. A final h_1-value can be found after several iterations. A value of $\alpha_1 = 1.04$ is typically used for steady-state uniform channels. The actual discharge is then found by multiplying the discharge computed above by C_d from Fig. 2 for the appropriate H_1/L-value.

The above method can be used to produce a rating table by using a very simple calculator. The error, X_c, in the listed discharges will be less than:

$$X_c = 2(21 - 20C_d) \tag{17}$$

If a computer is available, a rating table can be produced in which the error, X_c, is less than 2%. By using a mathematical model of flow through broad-crested weirs, which was developed by Replogle (1975), it is possible to match $Q/d^{2.5}g^{0.5}$ directly to h_1/d. This is because this model uses boundary layer drag concepts to determine the energy losses due to friction and a non-uniform velocity distribution. The model balances these friction losses between the values of H_1 and H_c.

It follows from eqs. 1 and 6 that there is a direct relationship between $Q/d^{2.5}g^{0.5}$ and h_1/d. Because of the complex geometries involved in the circular cross-sections it is convenient to use an empirical equation for this relationship to replace these two equations. A good fit of $Q/d^{2.5}g^{0.5}$ vs. h_1/d can be represented as a polynomial (Replogle, 1977a), or as a modified power function of the form suggested by Replogle et al. (1980):

$$Q/d^{2.5}g^{0.5} = C_e(h_1/d + K_h/d)^u \tag{18}$$

where C_e, K_h and u are constants in the empirical equation. This equation will be used in the remainder of this paper.

WEIR CALIBRATIONS

The development of the dimensionless form of the head—discharge equation is based on similitude, or more specifically, Froude's modeling law. However, the material roughness of a small or large pipe will probably be the same and thus the relative roughness will be different. Using earlier research data (Replogle, 1977b), the mathematical model (Replogle, 1975) was used to study the related scale effects for pipe diameters, d, ranging from 0.3 to 2.0 m, with sill heights, p_1, between 0.15d and 0.35d, while the values of h_1 ranged from the maximum permissible level $h_{1\,max} = 0.9d - p_1$ to 0.1 $h_{1\,max}$. The basis for this maximum level is discussed later. The throat length was limited by $h_{1\,max}/L \geqslant 0.7$ or $L \simeq (h_{1\,max})/0.7$. An absolute roughness height of 0.00015 m was used and the approach velocity coefficient, $\alpha_1 = 1.04$.

The computed discharges for given values of relative head, h_1/d, for pipe diameters from 0.5 to 2.0 m were extremely close and in general well within a range of ±0.2% for the full flow range. For the smaller diameter, $d =$ 0.30 m, the discharge began to deviate somewhat more, reflecting the increasing influence of relative roughness. For low relative heads the computed discharge differences, for 0.3—2.0-m diameters, were ±1.0% for $p_c/d =$ 0.15 and ±1.8% for $p_c/d = 0.35$. However, these computed differences were for the lowest permissible flows. For most of the flow range over these RBC broad-crested weirs the computed difference was less than ±0.5%. These results are not unexpected since for low heads and related flows the friction above the sill plays a greater role in determining the true discharge.

CURVE-FIT ROUTINE

Since all weir and pipe combinations can be considered as Froude models of each other, rating tables were developed for a diameter $d = 1.0$ m and for sill heights p_c, ranging from 0.05 to 0.50 m. These ratings were fit to eq. 18 with a curve-fit routine for complex functions (Kimball, 1971). This routine uses an iterative procedure to search for the optimum values of the coefficients of eq. 18.

The resulting values of C_e, K_h/d and u are given in Fig. 3. The equations reproduce the values computed by the mathematical model to within less than ±0.25% and most differences were on the order of ±0.1%.

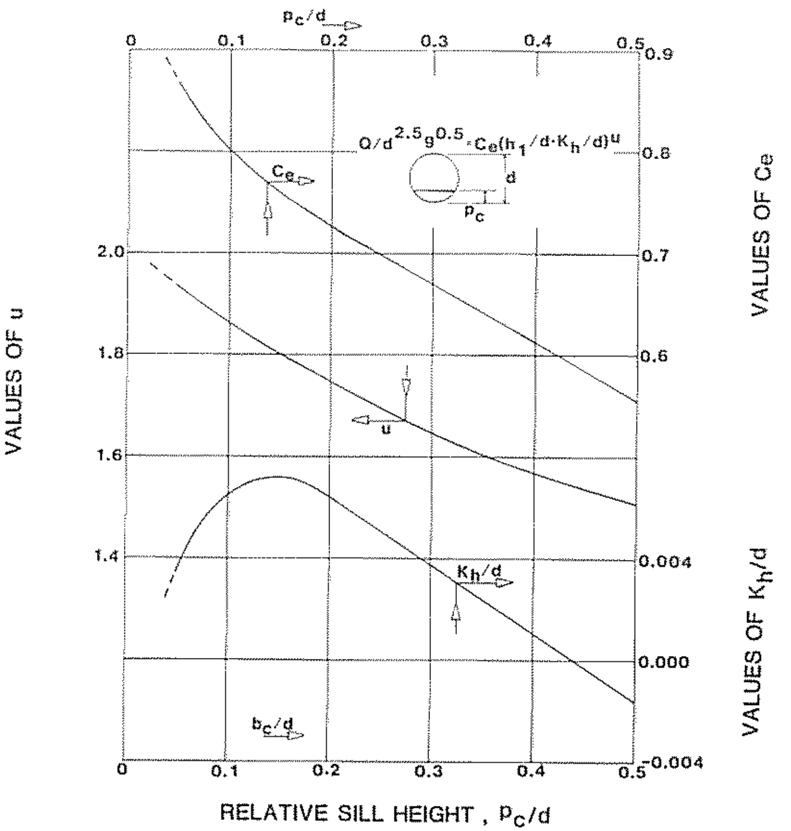


Fig. 3. Values of C_e, u and k_h/d for RBC broad-crested weirs in circular channels.

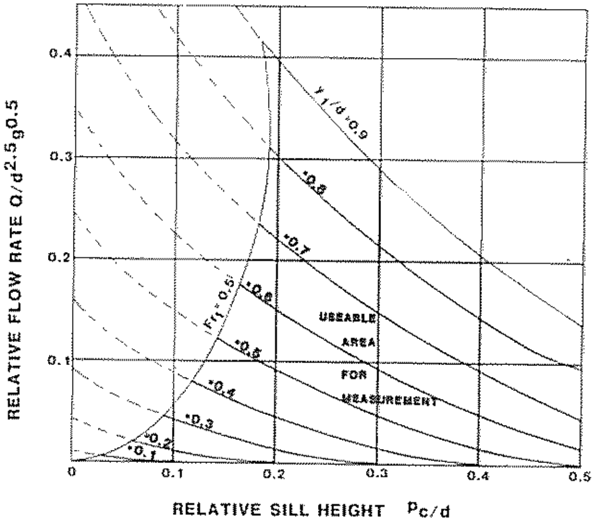


Fig. 4. Discharge capacity of the RBC broad-crested weirs.

MAXIMUM CAPACITY

Placing an obstruction in a pipe is likely to limit its capacity because the obstruction may increase the water depth to such an extreme that the pipe flows full. If this occurs there is no free water surface at the gauging station and the RBC broad-crested weirs cannot be used. This condition limits the applications to approximately:

$$y_1 = (p_1 + h_1) \leqslant 0.9d \tag{19}$$

A second limit of application arises from the demand to have a reasonably turbulent free water surface at the gauging station. For this to occur the Froude number should not exceed ~0.5. Both limits of application are shown in Fig. 4, illustrating the usable ranges for this type of weir.

HEAD-LOSS REQUIREMENT

The broad-crested weir, of all known critical-flow discharge measuring structures, measures the discharge as a function of upstream sill-referenced head accurately with the least head-loss requirement. This head loss can be written as:

$$\Delta H = H_1 - H_2 = H_1(1 - H_2/H_1) \tag{20}$$

In this equation, that limiting submergence ratio H_2/H_1 at which the discharge for the particular weir module is reduced less than 1% because of the tailwater level is called the modular limit. Bos (1976) developed a procedure for determining the modular limits and head-loss requirements for long-throated flumes. Upon refinement by Bos and Reinink (1981) this procedure, with minor modifications, was incorporated in the mathematical model for weir discharge and was used to determine the modular limits and head-loss requirements for the weirs given here. The values for modular limits ranged from ~0.65 for low flows over high sills to more than 0.95 for high flows over low sills.

The modular limit is effected by the downstream expansion ratio and the relative velocity of flow in the downstream channel (Bos and Reinink, 1981). We recommend an expansion ratio of 6 : 1 (horizontal to vertical distance) when head loss is to be minimized, and a rapid (vertical) expansion (0 :1) when head loss is not critical. The latter reduces the cost and complexity of the weir, particularly if it is placed at the end of a pipe as shown in Fig. 7. The gradual expansion can also be truncated when it reaches roughly one-half the sill height from the pipe invert.

The required head loss for maintaining modular flow over a structure with a rapid expansion as a function of upstream sill-referenced depth for several values of sill height is plotted in Fig. 5. Because of some uncertainty in modular limit calculations and uncertainty about actual flow conditions, we recommend that the modular limit not exceed 0.9 (equal to a head loss of $0.1H_1$) for truncated weirs, i.e. weirs without a downstream transition. As shown in Fig. 5, the computed head loss is used until the modular limit reaches 0.9. Then a head-loss value of $0.1H_1$ is used.

If the pipe is discharging into a reservoir or pool, the head loss will be

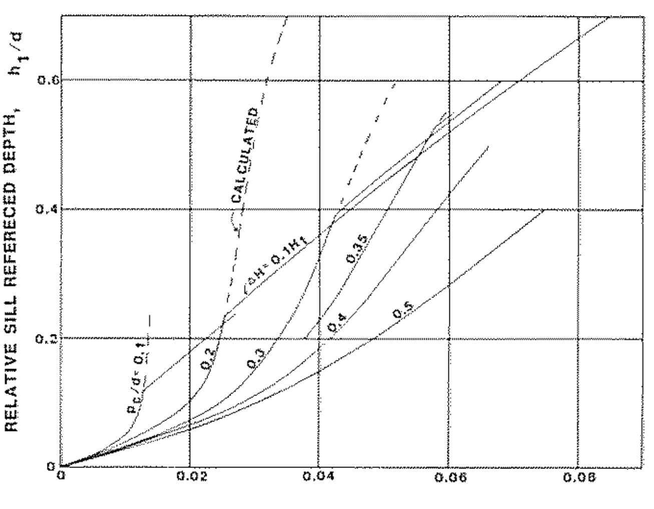


Fig. 5. Head loss required for maintaining modular flow over RBC broad-crested weirs with sudden expansion (for a weir placed in a continuous channel of constant cross-section with no sudden drop).

greater since there is less energy recovery. The modular limit for this case should be taken as 0.60 or $\Delta H_1 = 0.4H_1$. The addition of a downstream ramp can reduce the required head loss. A method for calculating the head loss for different combinations of tailwater conditions and ramp slopes is given in Bos and Reinink (1981).

DESIGN

The design of RBC broad-crested weirs is relatively straightforward. The weir sill must be high enough so that flow is modular and low enough to provide sufficient capacity. There are two situations of interest for designing a weir in a pipe. The weir can be placed somewhere in the middle of a straight pipe, or at the end of a pipe, such as the entrance to a deep manhole. The latter is probably more common because the site is more accessible. When the weir is placed in the interior of a pipe, the presence of the weir must cause a rise in the upstream water surface due to the required head loss. This increase in flow area upstream causes a proportional decrease in velocity and subsequent sediment deposition. Where this is a problem, a downstream ramp should be considered to reduce the head loss as much as possible. Because of the shape of these weirs, sediment problems are further aggravated at low flows since the free flowing (normal depth) water surface drops proportionally faster than the water surface upstream from the weir. Thus the velocity difference becomes greater and greater with decreasing discharge. For situations where wide fluctuations in flow rate exist and sedimentation is a problem, an alternative weir shape or location should be considered.

An alternative location for a measuring site is the end of a pipe, particularly where a drop in water surface exists. For long pipes where flow depth is controlled by channel friction, a weir can often be designed so that the water level upstream from the weir either matches or is below the normal water level in the pipe. In this way, we can considerably reduce the effects of the weir on sediment deposition.

If the weir is located in a section of pipe, the normal depth, y_n, equals y_2. Hence, at maximum flow:

$$y_2 + \Delta h \leqslant p_1 + h_1 \leqslant 0.9d \tag{21}$$

Eq. 21 gives the limits on design to provide for modular flow $y_1 \geqslant y_2 + \Delta h$ and to keep the pipe from flowing full $y_1 \leqslant 0.9d$ at maximum flow. If flow in the pipe is caused to occur by downstream backwater effects (in excess of normal depth), these criteria should be checked at low flows.

With a weir at the end of a pipe and sufficient overfall, the weir can be lower for better sediment transport. Preferably, now the normal depth, y_n, should be larger than or equal to y_1. Hence at maximum flow:

$$y_1 = p_1 + h_1 \leqslant 0.9d \tag{22}$$

EXAMPLE

The design procedure is illustrated by the following example for a weir in a pipe.

Given: A 0.75-m diameter pipe with a maximum discharge of:

$Q_{\text{design}} = 0.35\ \text{m}^3\ \text{s}^{-1}$ at $y_n = 0.60$ m deep

Required: Design a weir sill and select the parameters for eq. 18 applying to the selected sill for this pipe and discharge rate.

Solution:

(1) First estimate an approximate dimensionless sill-referenced depth, h_1/d. For this we need the dimensionless ratios for maximum design flow rate, Q, and normal depth, y_n:

$$Q_{\text{design}}/(d^{2.5}g^{0.5}) = 0.35/(0.75^{2.5} \times 9.81^{0.5}) = 0.229$$

and

$$y_n/d = 0.6/0.75 = 0.80$$

For this example we assume that the pipe itself is near enough to horizontal (less than 1% slope) that $p_1 \simeq p_c$. We next enter Fig. 4 with the above dimensionless Q-value of 0.229 and extending to the maximum flow depth curve of $y_1/h = 0.9$ to find the upper limit for the relative sill height. We find that:

$$p_c/d \leqslant 0.37$$

and since y_1/d must be at least 0.8 plus some head loss, we also read from the curve of $y_1/h = 0.8$, the lower limit on relative sill height,

$$p_c/d \geqslant 0.28$$

(2) Thus, try a relative sill height of $p_c/d = 0.30$, and again in Fig. 4 for relative Q of 0.229, read:

$$y_1/d \simeq 0.82$$

From the physical relations as illustrated in Fig. 1, calculate:

$$h_1/d = y_1/d - p_c/d = 0.82 - 0.30 = 0.52$$

(3) Now obtain an estimated minimum required head loss from Fig. 5 for the h_1/d calculated above and the p_c/d relative height curve for the selected value of 0.30, and find:

$$\Delta H/d = 0.058$$

(Note that the portion of the $p_c/d = 0.3$ curve used in this case is the straight-line part resulting from the 90% modular limit restriction recommended by the authors, i.e. $\Delta H = 0.1H_1$.)

(4) Next, improve the estimate of the minimum h_1/d-value needed. Using

the left-hand portion of eq. 21 rearranged in more convenient dimensionless form as:

$y_2/d + \Delta h/d \leqslant p_1/d + h_1/d$

and assuming the pipe is horizontal so that $y_2/d = y_n/d = 0.80$, and $p_1/d = p_c/d = 0.3$, and also that $\Delta H/d = 0.058$, calculate:

$0.80 + 0.058 \leqslant 0.3 + h_1/d$ thus $h_1/d \geqslant 0.558$

The relative sill-referenced head must be greater than 0.558 to maintain modular flow.

(5) Now compute the actual h_1/d-value from eq. 18 for comparison to the above limiting value. Enter Fig. 3 for $p_c/d = 0.3$ and obtain the parameters for eq. 18:

$C_e = 0.670;$ $u = 1.65;$ $k_h/d = 0.0037$

Thus for eq. 18:

$Q/(d^{2.5}g^{0.5}) = C_e(h_1/d + k_h/d)u$

substituting:

$0.229 = 0.67(h_1/d + 0.0037)^{1.65}$

and

$h_1/d = 0.518$

which is less than the required value of 0.558, and the modular limit will be exceeded. Therefore, select a higher sill.

(6) Try $p_c/d = 0.35$ and repeat steps (2)—(5):

Step (2′): for $p_c/d = 0.35$ and from Fig. 4:

$y_1/d \simeq 0.88$ and $h_1/d = 0.88 - 0.35 = 0.53$

Step (3′): From Fig. 5 for $h_1/d = 0.53$ and $p_c/d = 0.35$:

$\Delta H/d = 0.058$

Step (4′): Eq. 21:

$0.80 + 0.058 \leqslant 0.35 + h_1/d$ and $h_1/d \geqslant 0.508$

Step (5′): Fig. 3, for $p_c/d = 0.35$, for use in eq. 18:

$C_e = 0.640;$ $u = 1.61;$ $k_n/d = 0.0024$

compute h_1/d:

$0.229 = 0.64(h_1/d + 0.0024)^{1.61}$ and $h_1/d = 0.526$

which is greater than the required value from step (4′) above.

This design is satisfactory and will meet the restrictions on maximum pipe flow depth and modular limit.

(7) If the weir for the above situation were to be located at the end of a

pipe with a free overfall, then the limiting conditions described by eq. 22 would apply, with no concern for modular limit restrictions. Thus:

$$y_1/d = p_1/d + h_1/d \leqslant 0.9$$

Now, selecting a lower sill height such that normal depth, y_n, exists upstream at y_1, is also possible. Thus, entering Fig. 4 with the relative flow rate value of 0.229 and extending to the right to a relative depth, y_1/d, of 0.8, yields a relative sill height $p_c/d = 0.28$. Therefore, we recommend that selecting the next lower convenient value such as $p_c/d = 0.25$, which is still in the usable range of Fig. 4, will result in a suitable structure. Entering Fig. 3 for this value of p_c/d yields:

$$C_e = 0.695; \qquad u = 1.70; \qquad k_h/d = 0.0051$$

Thus from eq. 18:

$$0.229 = 0.695(h_1/d + 0.0051)^{1.70} \qquad \text{and} \qquad h_1/d = 0.515$$

Hence

$$y_1/d = h_1/d + p_c/d = 0.515 + 0.250 = 0.765$$

$$y_1 = 0.765d = 0.765 \times 0.75\,\text{m} = 0.574\,\text{m}$$

Since the original given flow depth was 0.600 m, this will produce a slight drawdown of 0.026 m which will also aid in bedload sediment movement.

(8) The final calibration equations for these two weirs are constructed from eq. 18.

For the original case: $C_e = 0.640$; $u = 1.61$; $k_h/d = 0.0024$.

Thus for:

$$Q/d^{2.5}g^{0.5} = C_e\,(h_1/d + k_h/d)^u$$

$$Q/(0.75^{2.5} \times 9.81^{0.5}) = 0.640(h_1/0.75 + 0.0024)^{1.61}$$

or

$$Q/1.5258 = 0.640(1/0.75)^{1.61}\,(h_1 + 0.0018)^{1.61}$$

$$Q = 1.552(h_1 + 0.0018)^{1.61}$$

with h_1 in m and Q in $m^3\,s^{-1}$.

For the free-overfall case: $C_e = 0.695$; $u = 1.70$; $k_h/d = 0.0051$

$$Q/1.5258 = 0.695(1/0.75)^{1.70}\,(h_1 + 0.0038)^{1.70}$$

$$Q = 1.73(h_1 + 0.0038)^{1.70}$$

with h_1 in m and Q in $m^3\,s^{-1}$.

Both equations and the normal depth vs. discharge curve of the pipe are plotted in Fig. 6. From this figure the backwater effect of both weirs can be derived. As illustrated, the low weir reduces the approach velocity from that at normal depth by 10% at 70% of Q_{design}, and is greater than the

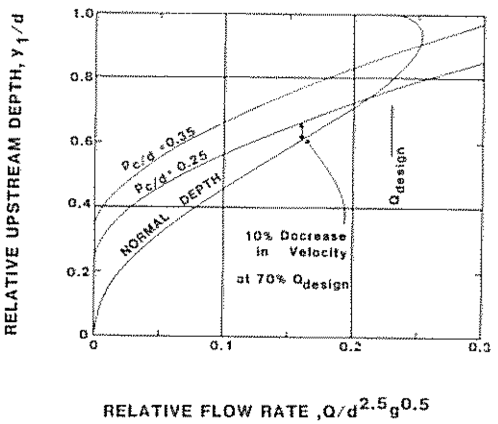


Fig. 6. Stage—discharge curves for design examples.

velocity at normal depth for Q_{design}. This is conducive for the passage of sediment.

For our first example, we can determine the sill width from Table II. Thus, for $p_c/d = 0.35$, $b_c/d = 0.9539$, and the constructed sill width should be:

$$b_c = 0.9539 \times 0.75\,\text{m} = 0.7154\,\text{m}$$

If, as constructed, b_c was actually 0.72 m ($b_c/d = 0.72/0.75 = 0.9600$) then we can use Table II to correct eq. 18 to find the corresponding value, $p_c/d = 0.36$ (Fig. 7). Since this value is also between the limiting values computed in step (1) of the example, it is suitable and a revised calibration equation should be obtained from eq. 18.

CONSTRUCTION

One of the advantages of the RBC broad-crested weir is its relative ease of construction. The weir can be constructed of concrete or a temporary structure can be made of sheet metal or waterproof Plywood®. Only one smooth finished surface is required; the truly horizontal weir sill. The ramp must make a distinct break with the sill; it can be hand-plastered and the pipe makes up the rest of the flume surface.

Also for pipes on a slight slope the weir sill must be horizontal. It is important that the sill width at the control section is as intended. It is stressed that the width b_c and thus the ratio b_c/d determines the head—discharge relation of the weir; not the derived (secondary) value p_c/d. Upon construction of the weir the exact value of b_c must be measured (Fig. 7) upon which the ratio p_c/d can be calculated by use of eqs. 10 and 11, or reference to Table II. Assuming $p_1 \neq p_c$ the final rating table for the weir can then be derived. If $p_1 \simeq p_c$, then the head (or discharge) must be adjusted

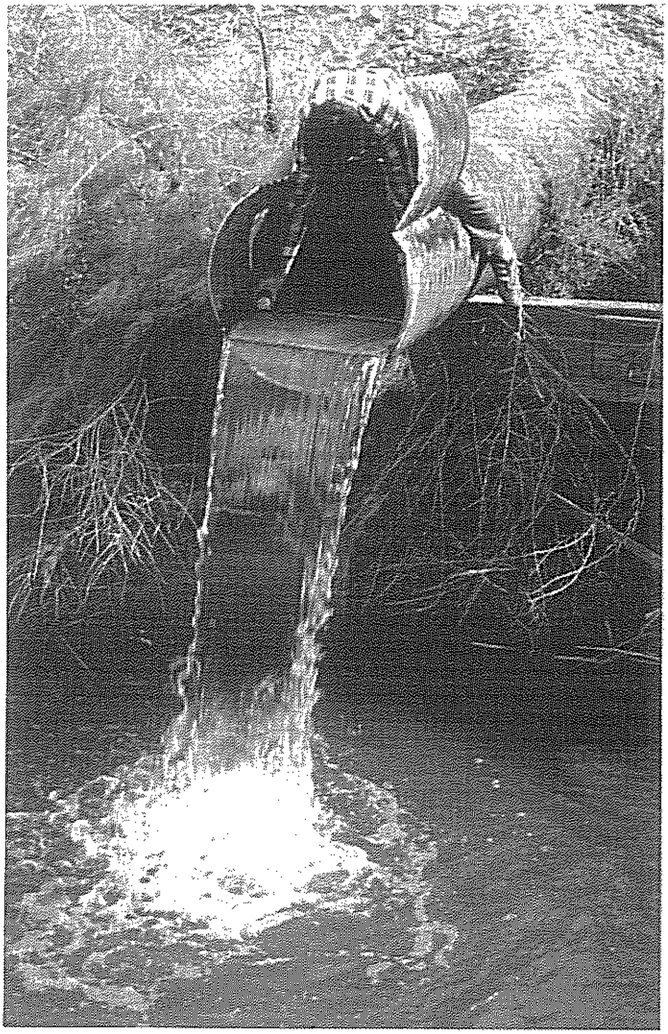


Fig. 7. Upon construction, the exact width, b_c, at the control section should be measured.

for the change in the velocity and the approach area from that assumed for $p_1 = p_c$.

Some slope of the pipe can be tolerated as long as it causes less than 1% variation in sill width throughout the length of the sill. Simple geometric considerations show that this criterion would appear to allow slopes that are quite steep if the sill height equals half the pipe diameter. However, the effective approach velocity would be changed significantly and would have to be considered in adjusting the rating curves. At low sill heights this 1% limitation would be exceeded at ~1% pipe slope. So, we recommend that pipes in excess of 1% slope will need rating and design considerations that are beyond the scope of this paper.

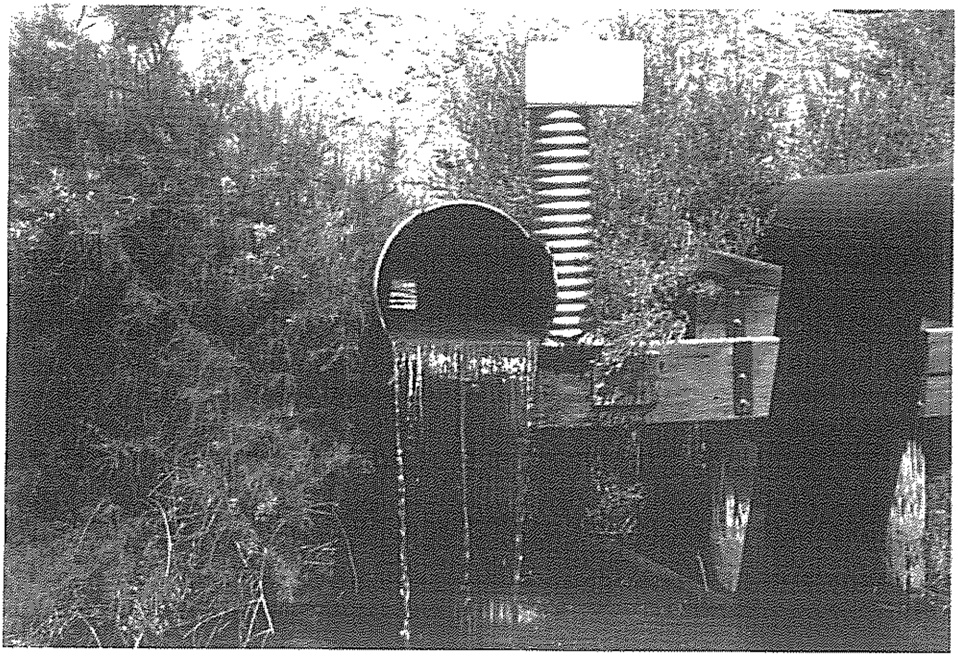


Fig. 8. Flow through an RBC weir can be monitored with a stilling well and recorder.

For this type of weir it is not practical to measure the sill-referenced head, h_1, in the pipe. The logical alternative is to make a pressure tap or install a sensing pipe at the gauging station and record the head from a clear plastic pipe mounted on the side of the pipe. Alternatively, a stilling well and related recorder can be installed (Fig. 8).

SUMMARY

The existing experience with and theory of long-throated flumes has led to the adaptation of the RBC broad-crested weir to flow in circular pipes flowing partly full. The ideal flow equations were solved and the resulting stage—discharge relationship has been listed in Table I. A mathematical model of flow over broad-crested weirs was used to determine the stage—discharge relationship for actual flow. These results were fit to an empirical equation in non-dimensional form for a wide range of sill heights and pipe sizes. The resulting constants have been plotted in Fig. 3. Also given are the maximum capacity and head loss requirements for any given weir size. Maximum calibration errors result from several sources including: basic calibration, ± 2%; effects of friction not accounted for by Froude modeling, ± 0.5%; effects of empirical curve fit, ± 0.5%; and errors in zero setting and depth detection, which varies with the size of flume. In general, the calibrations will be better than indicated above and should be well within ± 3%.

An example is given which shows how to design these weirs to minimize the effects of the weir on flow conditions in the channel. In many situations, these weirs can be designed to pass a majority of the sediment which moves through the channel.

These weirs are simple and inexpensive to construct and should be easily adapted to current measurement needs in circular open channels.

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