4 Broad-crested weirs

Classified under the term 'broad-crested weirs' are those structures over which the streamlines run parallel to each other at least for a short distance, so that a hydrostatic pressure distribution may be assumed at the control section. To obtain this condition, the length in the direction of flow of the weir crest (L) is restricted to the total upstream energy head over the crest (H_1). In the following sections the limitation on the ratio H_1/L will be specified for the following types of broad-crested weirs:

4.1 Horizontal broad-crested weir;
4.2 The Romijn movable measuring/regulating weir;
4.3 Triangular broad-crested weir;
4.4 Broad-crested rectangular profile weir;
4.5 Faiyum weir.

For details on other types of broad-crested weirs see Bos et al. (1984) and Bos (1985).

4.1 Horizontal broad-crested weir

4.1.1 Description

This weir is in use as a standard discharge measuring device and, as such, is described in the British Standard 3680, 1969, which is partly quoted below. The weir comprises a truly level and horizontal crest between vertical abutments. The upstream corner is rounded in such a manner that flow separation does not occur. Flow separation also can be avoided by using an upstream ramp which slopes between 2 – to – 1 and 3 – to – 1 (horz. to vert.). See Figure 1.34 for a longitudinal profile. This upstream sloping face is a cost-effective solution if the weir is constructed in concrete. Downstream of the horizontal crest there may be a vertical face or a downward slope, depending on the submergence ratio under which the weir should operate at modular flow.

The weir structure should be rigid and watertight and be at right angles to the direction of flow.

The dimensions of the weir and its abutments should comply with the requirements indicated in Figure 4.1. The minimum radius of the upstream rounded nose (r) is 0.11 H_{1max}, although for the economic design of field structures a value $r = 0.2\ H_{1max}$ is recommended. The length of the horizontal portion of the weir crest should not be less than 1.45 H_{1max}. To obtain a favourable (high) discharge coefficient (C_d) the crest length (L) should be close to the permissible minimum. In accordance with Section 2.2 the head measurement section should be located a distance of between two and three times H_{1max} upstream of the weir block.

4.1.2 Evaluation of discharge

According to Equation 1-37 Section 1.9.1, the basic stage-discharge equation for a broad-crested weir with a rectangular throat reads

$$Q = C_d C_v \frac{2}{3}\sqrt{\frac{2}{3}g}\ b_c h_1^{1.50} \qquad (4\text{-}1)$$

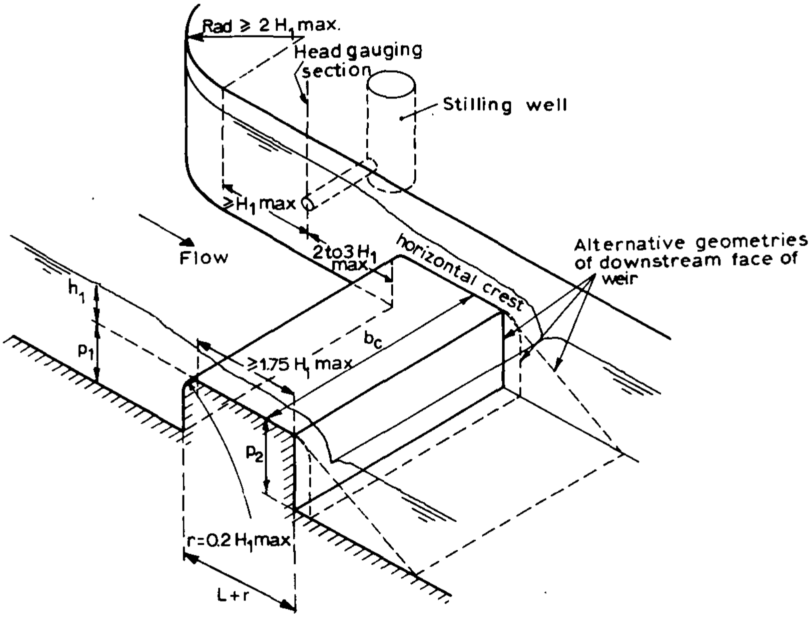


Figure 4.1 Dimensions of round-nose broad-crested weir and its abutments (adapted from British Standards Institution 1969)

For water of ordinary temperatures, the discharge coefficient (C_d) is a function of the upstream sill-referenced energy head (H_1), and the length of the weir crest in the direction of flow (L). It can be expressed by the equation (Bos 1985)

$$C_d = 0.93 + 0.10\,H_1/L \qquad (4\text{-}2)$$

The appropriate value of the approach velocity coefficient (C_v) can be read from Figure 1.12 (Chapter 1).

The error in C_d of a well maintained broad-crested weir, which has been constructed with reasonable care and skill, can be deduced from the equation (Bos 1985).

$$X_c = \pm(3\,|H_1/L - 0.55|^{1.5} + 4) \text{ per cent} \qquad (4\text{-}3)$$

The method by which this error is to be combined with other sources of error is shown in Annex 2.

Table 4.1 gives a series of rating tables for rectangular weirs. The groupings of weir width were selected to keep the error due to the effects of the sidewalls to less than 1%. Ratings are given for a number of sill heights to aid in design. Discharges in these tables are limited to keep the approach channel Froude number below 0.45. Interpolation between sill heights will give reasonable results. If the approach area is larger than that used to develop these rating tables, either because of a higher sill or a wider approach, the ratings must be adjusted for C_v (see Figure 1.12). To simplify this process, the discharge over the weir for a C_v value of 1.0 is given in the far right column of each grouping. This discharge column is labeled as $p_1 = \infty$, since for $C_v = 1.0$ the velocity of approach is zero, as would be the case if the weir were the outlet

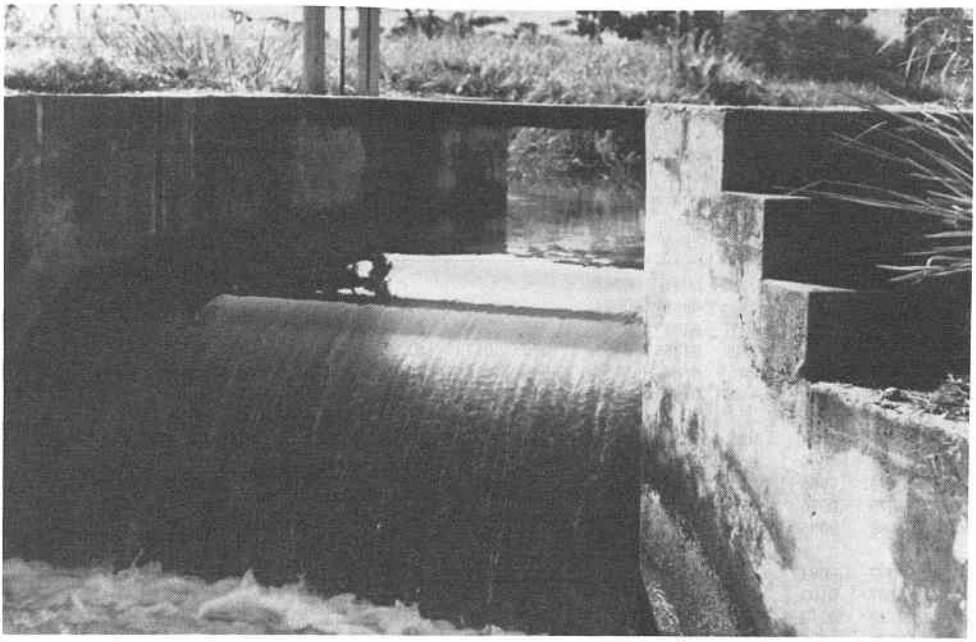


Photo 1 Downstream view of a broad-crested weir

of a deep reservoir or lake. Under this circumstance, the weir has the lowest discharge for a given upstream head. Note that at the very low heads, the discharge for the weirs with rectangular approach channels approaches $p_1 = \infty$ because the approach velocities are small.

The ratings given in Table 4.1 are for the throat lengths L given at the head of each group columns. When the maximum design discharge of a structure is much less than the maximum discharge shown in the rating table, the aforementioned throat length may be longer than necessary. A value of $L = 1.5\ H_{1\max}$ is a reasonable compromise between providing a long enough throat to avoid the effects of streamline curvature and minimizing the size of the structure. The throat length may be reduced to this value provided that it does not become shorter than about two-thirds of the L value in the table heading. Such a length reduction causes the weir discharge to increase by less than 1%. The length of the converging transition L_b should be between 2 and 3 times p_1. The distance between the gauging station and the start of the throat $(L_a + L_b)$ should be between 2 and 3 times $H_{1\max}$, and the distance between the gauging station and the start of the converging transition L_a should be greater than $H_{1\max}$.

Table 4.1 Rating Tables for rectangular Weirs in Metric Units with Discharge per Meter Width*

0.10 ≤ b_c ≤ 0.20 m L = 0.2 m			0.20 ≤ b_c ≤ 0.30 m L = 0.35 m			0.30 ≤ b_c ≤ 0.50 m L = 0.5 m				0.5 ≤ b_c ≤ 1.0 m L = 0.75 m				
h_l (m)	q (m^3/s per meter width)		h_l (m)	q (m^3/s per meter width)		h_l (m)	q (m^3/s per meter width)			h_l (m)	q (m^3/s per meter width)			
	p_l = 0.05 m	p_l = ∞		p_l = 0.1 m	p_l = ∞		p_l = 0.1 m	p_l = 0.2 m	p_l = ∞		p_l = 0.1 m	p_l = 0.2 m	p_l = 0.3 m	p_l = ∞
			.025	.0064	.0063					.050	.0186	.0183	.0182	.0181
			.030	.0085	.0084					.055	.0216	.0212	.0210	.0209
.014	.0026	.0026	.035	.0108	.0107	.035	.0108	.0106	.0106	.060	.0248	.0242	.0240	.0239
.016	.0032	.0032	.040	.0133	.0131	.040	.0133	.0131	.0130	.065	.0281	.0274	.0272	.0270
.018	.0039	.0038	.045	.0160	.0157	.045	.0160	.0157	.0156	.070	.0316	.0308	.0305	.0303
.020	.0046	.0045	.050	.0189	.0184	.050	.0188	.0185	.0183	.075	.0352	.0342	.0339	.0336
.022	.0054	.0053	.055	.0220	.0213	.055	.0219	.0214	.0212	.080	.0390	.0378	.0374	.0371
.024	.0062	.0060	.060	.0252	.0244	.060	.0251	.0245	.0242	.085	.0429	.0416	.0411	.0407
.026	.0070	.0068	.065	.0285	.0275	.065	.0285	.0278	.0274	.090	.0470	.0454	.0449	.0444
.028	.0079	.0076	.070	.0321	.0308	.070	.0320	.0312	.0307	.095	.0512	.0494	.0488	.0482
.030	.0088	.0085	.075	.0357	.0342	.075	.0357	.0347	.0341	.100	.0555	.0535	.0528	.0521
.032	.0097	.0094	.080	.0396	.0377	.080	.0395	.0383	.0376	.105	.0600	.0577	.0570	.0561
.034	.0107	.0103	.085	.0435	.0414	.085	.0435	.0421	.0412	.110	.0646	.0621	.0612	.0602
.036	.0117	.0112	.090	.0476	.0451	.090	.0476	.0460	.0450	.115	.0693	.0665	.0656	.0644
.038	.0128	.0122	.095	.0519	.0490	.095	.0519	.0500	.0488	.120	.0742	.0711	.0700	.0688
.040	.0138	.0132	.100	.0563	.0529	.100	.0561	.0540	.0528	.125	.0792	.0758	.0746	.0732
.042	.0150	.0142	.105	.0608	.0570	.105	.0606	.0583	.0567	.130	.0843	.0806	.0793	.0776
.044	.0161	.0153	.110	.0655	.0611	.110	.0652	.0626	.0608	.135	.0896	.0855	.0840	.0822
.046	.0173	.0164	.115	.0702	.0654	.115	.0700	.0671	.0651	.140	.0949	.0905	.0889	.0869
.048	.0185	.0175	.120	.0752	.0697	.120	.0748	.0717	.0694	.145	.1004	.0956	.0939	.0916
.050	.0197	.0186	.125	.0802	.0741	.125	.0798	.0764	.0738	.150	.1061	.1009	.0989	.0965
.052	.0210	.0197	.130	.0854	.0787	.130	.0850	.0812	.0783	.155	.1118	.1062	.1041	.1014
.054	.0223	.0209	.135	.0907	.0833	.135	.0902	.0861	.0828	.160	.1176	.1116	.1094	.1064
.056	.0236	.0221	.140	.0961	.0880	.140	.0956	.0911	.0875	.165	.1236	.1172	.1147	.1115
.058	.0250	.0233	.145	.1017	.0928	.145	.1011	.0962	.0923	.170	.1297	.1228	.1202	.1166
.060	.0264	.0245	.150	.1074	.0977	.150	.1067	.1014	.0971	.175	.1359	.1285	.1257	.1219
.062	.0278	.0257	.155	.1132	.1026	.155	.1125	.1068	.1020	.180	.1422	.1344	.1314	.1272
.064	.0293	.0270	.160	.1191	.1077	.160	.1183	.1122	.1070	.185	.1486	.1403	.1371	.1325
.066	.0307	.0283	.165	.1251	.1128	.165	.1243	.1177	.1121	.190	.1552	.1464	.1430	.1380
.068	.0322	.0296	.170	.1312	.1180	.170	.1304	.1234	.1173	.195	.1618	.1525	.1489	.1435
.070	.0338	.0309	.175	.1375	.1233	.175	.1366	.1291	.1225	.200	.1686	.1587	.1549	.1492
.072	.0353	.0323	.180	.1439	.1286	.180	.1429	.1349	.1278	.210**	.1824	.1715	.1671	.1606
.074	.0369	.0337	.185	.1504	.1340	.185	.1493	.1409	.1332	.220	.1967	.1846	.1798	.1723
.076	.0385	.0350	.190	.1567	.1396	.190	.1559	.1469	.1387	.230	.2113	.1981	.1927	.1843
.078	.0402	.0365	.195	.1633	.1451	.195	.1625	.1530	.1442	.240	.2264	.2119	.2060	.1965
.080	.0419	.0379	.200	.1701	.1508	.200	.1693	.1593	.1498	.250	.2419	.2262	.2197	.2090
.082	.0436	.0393	.205	.1770	.1565	.205	.1762	.1656	.1555	.260	.2578	.2407	.2336	.2217
.084	.0453	.0408	.210	.1840	.1623	.210	.1831	.1720	.1612	.270	.2741	.2557	.2479	.2348
.086	.0470	.0423	.215	.1911	.1681	.215	.1902	.1786	.1671	.280	.2908	.2709	.2625	.2480
.088	.0488	.0438	.220	.1983	.1741	.220	.1974	.1852	.1730	.290	.3078	.2866	.2775	.2610
.090	.0506	.0453	.225	.2056	.1801	.225	.2047	.1919	.1789	.300	.3253	.3025	.2927	.2752
.092	.0524	.0468	.230	.2130	.1861	.230	.2121	.1987	.1849	.310	.3431	.3188	.3083	.2892
.094	.0543	.0484	.235	.2205	.1923	.235	.2196	.2056	.1910	.320	.3613	.3355	.3242	.3034
.096	.0562	.0499				.240	.2272	.2125	.1972	.330	.3799	.3524	.3404	.3178

Table 4.1 (continued)

$0.10 \leqslant b_c \leqslant 0.20$ m L = 0.2 m			$0.20 \leqslant b_c \leqslant 0.30$ m L = 0.35 m			$0.30 \leqslant b_c \leqslant 0.50$ m L = 0.5 m				$0.5 \leqslant b_c \leqslant 1.0$ m L = 0.75 m				
h_1 (m)	q (m^3/s per meter width)		h_1 (m)	q (m^3/s per meter width)		h_1 (m)	q (m^3/s per meter width)			h_1 (m)	q (m^3/s per meter width)			
	p_1 = 0.05 m	p_1 = ∞		p_1 = 0.1 m	p_1 = ∞		p_1 = 0.1 m	p_1 = 0.2 m	p_1 = ∞		p_1 = 0.1 m	p_1 = 0.2 m	p_1 = 0.3 m	p_1 = ∞
.098	.0581	.0515				.245	.2349	.2196	.2034	.340	.3988	.3697	.3568	.3325
.100	.0600	.0531				.250	.2427	.2268	.2097	.350	.4181	.3873	.3736	.3473
.105**	.0649	.0571				.260**	.2587	.2414	.2225	.360	.4378	.4053	.3907	.3624
.110	.0700	.0613				.270	.2750	.2563	.2355	.370		.4235	.4081	.3777
.115	.0753	.0656				.280	.2917	.2716	.2488	.380		.4421	.4258	.3932
.120	.0806	.0699				.290	.3088	.2872	.2623	.390		.4610	.4438	.4089
.125	.0861	.0744				.300	.3262	.3032	.2760	.400		.4802	.4620	.4248
.130	.0918	.0789				.310	.3441	.3195	.2900	.410		.4998	.4806	.4409
						.320	.3623	.3361	.3042	.420		.5196	.4994	.4573
						.330	.3808	.3531	.3186	.430		.5397	.5185	.4738
										.440		.5601	.5379	.4905
										.450		.5809	.5576	.5074
										.460		.6019	.5776	.5245
										.470		.6232	.5978	.5418
										.480		.6448	.6183	.5593
										.490		.6667	.6391	.5769
										.500		.6888	.6601	.5948
ΔH =	0.012 m or $0.1H_1$		ΔH =	0.025 m or $0.1H_1$		ΔH =	0.027 m or $0.1H_1$	0.044 m		ΔH =	0.028 m or $0.1H_1$	0.048 m or $0.1H_1$	0.063 m	

* L_b = 2 or 3 times p_1; $L_a \geqslant H_{1max}$; $L_a + L_b \geqslant$ 2 to 3 times H_{1max}.

** Change in head increment

(continued)

Table 4.1 (continued)

$1.0 \leqslant b_c \leqslant 2.0\,m$
$L = 1.0\,m$

h_1 (m)	q (m^3/s per meter width)			
	$p_1 =$ 0.2 m	$p_1 =$ 0.3 m	$p_1 =$ 0.4 m	$p_1 =$ ∞
.070	.0304	.0301	.0300	.0298
.080	.0374	.0370	.0369	.0298
.090	.0450	.0445	.0442	.0439
.100	.0531	.0524	.0521	.0516
.110	.0616	.0608	.0604	.0597
.120	.0706	.0696	.0691	.0683
.130	.0801	.0788	.0782	.0771
.140	.0900	.0885	.0877	.0864
.150	.1004	.0985	.0976	.0960
.160	.1112	.1090	.1079	.1059
.170	.1224	.1198	.1185	.1161
.180	.1339	.1319	.1295	.1267
.190	.1459	.1426	.1408	.1375
.200	.1583	.1545	.1525	.1487
.210	.1711	.1668	.1646	.1601
.220	.1842	.1794	.1769	.1718
.230	.1977	.1924	.1896	.1838
.240	.2116	.2058	.2027	.1961
.250	.2259	.2194	.2160	.2086
.260	.2405	.2334	.2297	.2214
.270	.2542	.2477	.2436	.2344
.280	.2708	.2624	.2579	.2477
.290	.2864	.2774	.2725	.2612
.300	.3024	.2927	.2873	.2749
.310	.3188	.3083	.3025	.2889
.320	.3355	.3242	.3180	.3032
.330	.3525	.3404	.3337	.3176
.340	.3698	.3569	.3498	.3323
.350	.3875	.3738	.3661	.3472
.360	.4055	.3909	.3828	.3623
.370	.4238	.4083	.3997	.3776
.380	.4424	.4261	.4168	.3931
.390	.4614	.4441	.4343	.4088
.400	.4806	.4624	.4520	.4248
.410	.5002	.4810	.4701	.4409
.420	.5200	.4999	.4883	.4573
.430	.5401	.5190	.5069	.4738
.440	.5607	.5385	.5257	.4905
.450	.5815	.5582	.5447	.5075

$b_c \geqslant 2.0\,m$
$L = 1.0\,m$

h_1 (m)	q (m^3/s per meter width)			
	$p_1 =$ 0.2 m	$p_1 =$ 0.4 m	$p_1 =$ 0.6 m	$p_1 =$ ∞
.100	.0521	.0511	.0508	.0506
.120	.0695	.0680	.0675	.0671
.140	.0889	.0866	.0858	.0852
.160	.1099	.1067	.1056	.1046
.180	.1326	.1283	.1268	.1253
.200	.1596	.1513	.1493	.1473
.220	.1827	.1756	.1732	.1704
.240	.2101	.2013	.1982	.1946
.260	.2389	.2283	.2245	.2199
.280	.2691	.2565	.2519	.2461
.300	.3008	.2859	.2805	.2733
.320	.3337	.3165	.3101	.3015
.340	.3681	.3483	.3409	.3306
.360	.4037	.3812	.3727	.3606
.380	.4406	.4153	.4056	.3914
.400	.4788	.4505	.4395	.4231
.420	.5182	.4868	.4744	.4556
.440	.5588	.5241	.5103	.4889
.460	.6007	.5626	.5472	.5229
.480	.6437	.6020	.5851	.5577
.500	.6878	.6425	.6239	.5932
.520	.7331	.6840	.6636	.6295
.540	.7796	.7265	.7042	.6664
.560	.8271	.7699	.7458	.7041
.580	.8758	.8144	.7884	.7425
.600	.9527	.8600	.8319	.7815
.620	.9765	.9063	.8762	.8212
.640	1.028	.9537	.9214	.8615
.660	1.081	1.002	.9674	.9025
.680	1.135	1.051	1.014	.9441
.700	1.191	1.101	1.062	.9864
.720		1.153	1.111	1.029
.740		1.205	1.160	1.073
.760		1.257	1.210	1.117
.780		1.311	1.262	1.161
.800		1.366	1.314	1.207
.820		1.422	1.367	1.252
.840		1.478	1.420	1.299
.860		1.535	1.474	1.346
.880		1.593	1.530	1.393
.900		1.652	1.586	1.441

Table 4.1 (continued)

1.0 ⩽ b_c ⩽ 2.0 m L = 1.0 m					b_c ⩾ 2.0 m L = 1.0 m				
h_1 (m)	q (m^3/s per meter width)				h_1 (m)	q (m^3/s per meter width)			
	p_1 = 0.2 m	p_1 = 0.3 m	p_1 = 0.4 m	p_1 = ∞		p_1 = 0.2 m	p_1 = 0.4 m	p_1 = 0.6 m	p_1 = ∞
.460	.6025	.5782	.5641	.5246	.920		1.712	1.642	1.490
.470	.6238	.5984	.5837	.5419	.940		1.773	1.700	1.539
.480	.6455	.6189	.6035	.5594	.960		1.834	1.758	1.588
.490	.6674	.6398	.6236	.5771	.980		1.897	1.817	1.638
.500	.6896	.6608	.6440	.5950	1.000		1.960	1.877	1.689
.510	.7122	.6822	.6646	.6130					
.520	.7350	.7038	.6855	.6312					
.530	.7580	.7257	.7065	.6496					
.540	.7814	.7478	.7279	.6682					
.550	.8050	.7702	.7495	.6869					
.560	.8290	.7929	.7715	.7059					
.570	.8532	.8158	.7936	.7249					
.580	.8776	.8390	.8159	.7442					
.590	.9024	.8624	.8385	.7636					
.600	.9274	.8861	.8613	.7832					
.610	.9527	.9102	.8844	.8029					
.620	.9782	.9343	.9077	.8228					
.630	1.004	.9588	.9312	.8429					
.640	1.030	.9835	.9550	.8632					
.650	1.056	1.008	.9790	.8836					
.660	1.083	1.034	1.003	.9041					
.670	1.110	1.059	1.028	.9249					
ΔH =	0.046 m or $0.1H_1$	0.066 m or $0.1H_1$	0.086 m		ΔH =	0.047 m or $0.1H_1$	0.087 m or $0.1H_1$	0.124 m or $0.1H_1$	

* L_b = 2 or 3 times p_1; $L_a \geqslant H_{1max}$; $L_a + L_b \geqslant$ 2 to 3 times H_{1max}

** Change in head increment

4.1.3 Modular limit

The flow over a weir is modular when it is independent of variations in tailwater level. For this to occur, assuming subcritical conditions in the tailwater channel, the tailwater energy level (H_2) must not rise beyond a certain percentage of the upstream energy head over the weir crest (H_1). Hence, the height of the weir above the bottom of the tailwater channel (p_2) should be such that the weir operates at modular flow at all discharges. The modular limit can be read from Figure 4.2 as a function of H_1/p_2 and the slope of the back face of the weir. A more accurate design value of p_2 may be established by the method persented in Section 1.15.

4.1.4 Limits of application

a. The practical lower limit of h_1 is related to the magnitude of the influence of fluid properties, to the boundary roughness, and to the accuracy with which h_1 can be determined. The recommended lower limit is 0.06 m or 0.05 times L, whichever is greater.
b. The limitations on H_1/p_1 arise from difficulties experienced when the Froude number $Fr_1 = v_1/(gA_1/B_1)^{0.5}$ in the approach channel exceeds 0.45.
c. The limitations on H_1/L arise from the necessity of ensuring a sensible hydrostatic pressure distribution at the critical section of the crest and of preventing the formation of undulations above the weir crest. Values of the ratio H_1/L should therefore range between 0.08 and 0.7.
d. The breadth (b_c) of the weir crest should not be less than L/5.

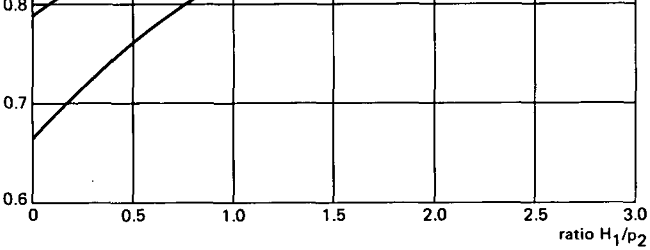


Figure 4.2 The modular limit as a function of H_1/p_2

4.2 The Romijn movable measuring/regulating weir

4.2.1 Description

The Romijn weir was developed by the Department of Irrigation in Indonesia as a regulating and measuring device for use in relatively flat irrigated regions where the water demand is variable because of different requirements during the growing season and because of crop rotation. A description of the weir was published in 1932 by Romijn, after whom the structure is named.

The telescoping Romijn weir consists of two sliding blades and a movable weir which are mounted in a steel guide frame:

a. the bottom slide is blocked in place under operational conditions and acts as a bottom terminal for the movable weir

b. the upper slide is connected to the bottom slide by means of two steel strips placed in the frame grooves and acts as a top terminal for the movable weir;

c. the movable weir is connected by two steel strips to a horizontal lifting beam. The weir crest is horizontal perpendicular to the flow and slopes 1-to-25 upward in the direction of flow. Its upstream nose is rounded off in such a way that flow separation does not occur. The operating range of the weir equals the maximum upstream head (h_1) which has been selected for the dimensioning of the regulating structure (see Figure 4.3).

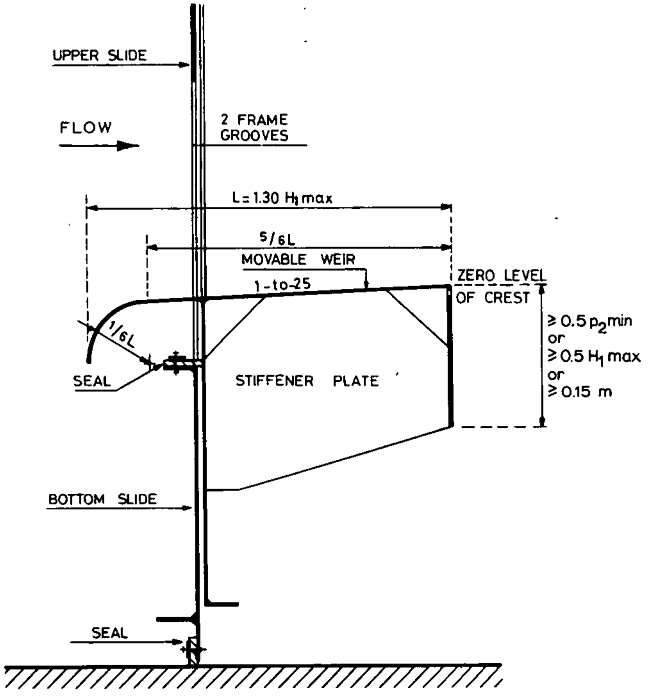


Figure 4.3 The Romijn movable weir

Although the Romijn weir has been included in this chapter on broad-crested weirs, from a purely hydraulic point of view this is not quite correct. Above the 1-to-25 sloping weir crest the streamlines are straight but converging so that the equipotential lines are curved. At the same time, the control section is situated more towards the end of the crest than if the crest were truly horizontal. Therefore, the degree of downward curvature of the overflowing nappe has a significant influence on the C_d-value.

To prevent the formation of a relatively strong eddy beneath the weir crest and the overflowing nappe, the weir should have a vertical downstream face. The reason for this is that especially under submerged flow conditions the nappe will deflect upwards due to the horizontal thrust of the eddy, resulting in up to 7% lower weir flows. The downstream weir face, which breaks the force of the eddy should have a minimum height of 0.5 p_{2min} or 0.5 H_{1max} or 0.15 m, whichever is greater.

As mentioned, the bottom slide, and thus the upper slide, is blocked in place during normal flow conditions. However, to flush sediments that have collected upstream of the weir, both slides can be unlocked and raised by moving the weir crest upward. After flushing operations the slides are pushed in place again by lowering the weir crest. To discourage misuse of the weir, the maximum flow capacity beneath the lifted bottom gate must be less than the flow over the weir in its lowest position. For this to occur, the travel of the upper gate is restricted so that the bottom gate cannot be lifted higher than 0.5 H_{1max} above the approach channel bottom.

The weir abutments are vertical and are rounded in such a way that flow separation does not occur. A rectangular approach channel is formed to assure an even flow distribution. The upstream head over the weir, h_1, is measured in this approach channel at a distance of between two and three times H_{1max} upstream of the weir face. The dimensions of the abutment should comply with the requirements indicated in Figure 4.4. The radius of the upstream rounding-off of the abutments may be reduced to $r \geqslant H_{1max}$ if the centre line of the weir structure is parallel to or coincides with the centre line of the undivided supply canal (in-line structure) or if the water is drawn direct from a (storage) basin.

If several movable weirs are combined in a single structure, intermediate piers should be provided so that two-dimensional flow is preserved over each weir unit, allowing the upstream head over the weir to be measured independently per unit. The parallel section of the pier should therefore commence at a point located at a distance of H_{1max} upstream of the head measurement station and extend to the downstream edge of the weir crest. Piers should have streamlined noses, i.e. of semi-circular or tapered semi-elliptical profile (1-to-3 axis). To avoid extreme velocity differences over short distances, the thickness of the intermediate piers should be equal to or more than 0.65 H_{1max}, with a minimum of 0.30 m.

Since the weir crest moves up and down, a fixed staff gauge at the head measurement station does not provide a value for the upstream head over the crest unless the weir crest elevation is registered separately in terms of gauged head. To avoid this procedure, the weir is equipped with a gauge that moves up and down with the weir crest (see Fig. 4.4). Zero level of this gauge coincides with the downstream edge of the weir crest, so that the upstream head over the crest equals the immersed depth of the gauge and can be read without time lag. The movable gauge is attached to the extended lifting beam as shown in Figure 4.7.

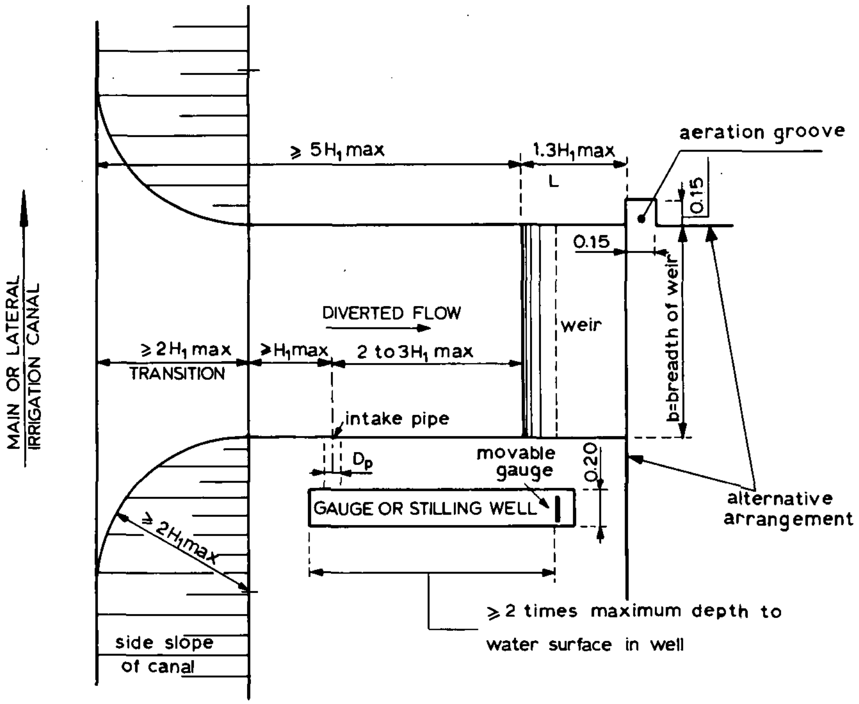


Figure 4.4 Hydraulic dimensions of weir abutments

4.2.2 Evaluation of discharge

According to Equation 1-37, Section 1.9.1, the basic head discharge equation for a broad-crested weir with a rectangular control section reads

$$Q = C_d C_v \frac{2}{3}\left[\frac{2}{3} g\right]^{0.50} b_c\, h_1^{1.50} \qquad (4\text{-}4)$$

Values of the discharge coefficient C_d may be read from Figure 4.5 as a function of the ratio H_1/L.

Since the weir crest height above the approach channel bed (p_1) is variable and to a certain extent independent of the head over the weir crest h_1, the approach velocity cannot be predicted unless p_1 is known. Engineers therefore tend to use either a constant C_d-value of 1.055 for all values of H_1/L or use Figure 4.5 to determine C_d by assuming that $h_1 \simeq H_1$.

Values for the approach velocity coefficient C_v may be read from Figure 1.12 as a function of the dimensionless ratio $C_d h_1/(h_1 + p_1)$, where p_1 is the variable height of the movable weir crest above the bottom of the rectangular approach channel. Over the range of p_1-values, an average C_v-value may be used in Equation 4-4 (see also Figure 4.8).

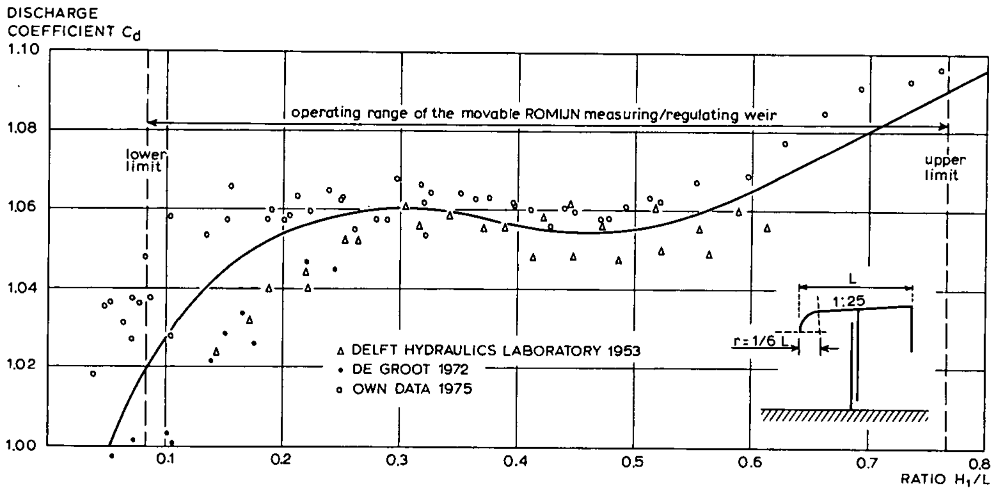


Figure 4.5 Values of C_d as a function of H_1/L for the Romijn weir

If a movable Romijn weir has been constructed and installed with reasonable care and skill, its discharge coefficient C_d may be expected to have an error of less than 3%. If an average value of $C_d = 1.055$ is used for all ratios of H_1/L, this C_d-values may be expected to have an error of less than 4%. To obtain these accuracies the weir should be properly maintained. The error in the C_v-coefficient depends on the minimum value of p_1 and the operating range of the movable weir. For the two most common weir types the error in C_v may be obtained from Section 4.2.4 and Figure 4.8. The method by which the coefficient errors have to be combined with other sources of error is shown in Annex 2.

4.2.3 Modular limit

In order to obtain modular flow the submergence ratio H_2/H_1 for which the modular discharge is reduced by 1% owing to the increasing tailwater level, should not exceed 0.30.

Results of laboratory tests have shown that the drowned flow reduction factor, and thus the modular limit, depends on a number of factors, such as the value of the ratio H_1/L and the crest height above the tailwater channel bottom, p_2. Since most energy loss occurs in the bottom eddy immediately downstream of the weir crest, little or no influence on the modular limit was observed if the side walls of the weir either terminated abruptly or flared under 1-to-6. Values of the average drowned flow reduction factor, f, (i.e. the factor whereby the equivalent modular discharge is decreased due to submergence) varies with H_2/H_1 as shown in Figure 4.6.

To prevent underpressure beneath the nappe influencing the discharge, the air pocket beneath the nappe should be fully aerated, for example by means of the two aeration grooves as shown in Figure 4.4.

4.2.4 Commonly used weir dimensions

The reader will have noted that all dimensions of both the weir and its abutments are related to the maximum value selected for the total energy head over the weir crest (H_{1max}). The loss of head required for modular flow is also related to the total energy head as $\Delta h = h_1 - h_2 \geqslant 0.70\ H_{1max}$.

Since the limiting factor in most relatively flat irrigated areas is the available head for open canal and weir flow, the maximum value of h_1 is limited to a certain practical value which approximates 0.45 m. The length of the weir crest in the direction of flow consequently equals L = 0.60 m, of which 0.50 m is straight and sloping 1 to 25 upward in the direction of flow and the remaining 0.10 m forms the rounded nose, its radius also being 0.10 m.

Theoretically any weir breadth greater or equal to 0.30 m may be used but to obtain a degree of standardization in the structures of an irrigation project a limited number of breadths should be employed. It is often practicable to use a breadth not greater than b_c = 1.50 m, since a central handwheel can then be used to move the weir while the groove arrangement can be a relatively simple one consisting of steel blades sliding in narrow (0.01 m) grooves. If the breadth b_c exceeds 1.50 m, a groove arrangement as shown in Section 6.5.1 may be used.

Examples of constructional drawings are shown in Figure 4.7.

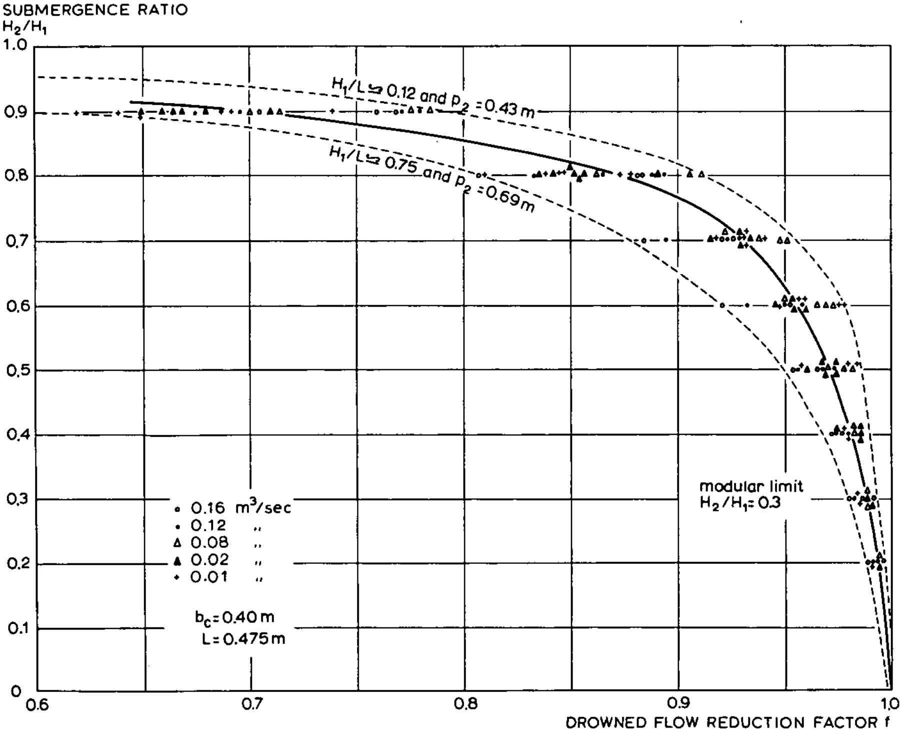
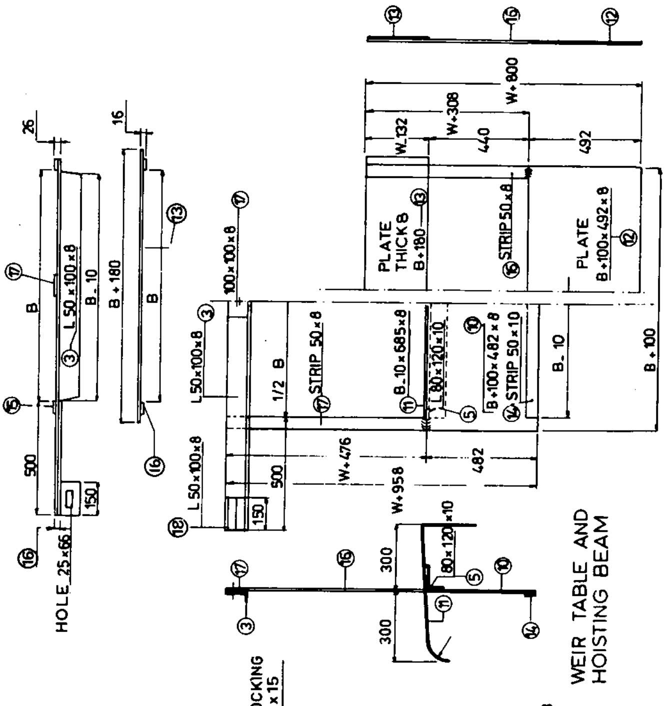
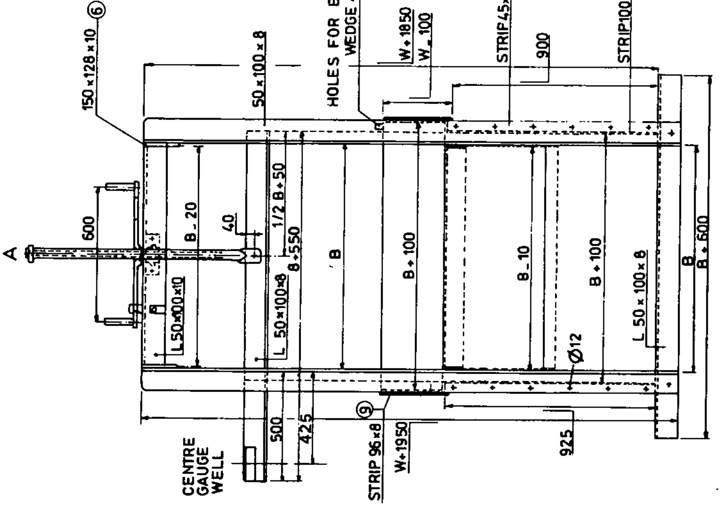


Figure 4.6 Drowned flow reduction factor for Romijn weir

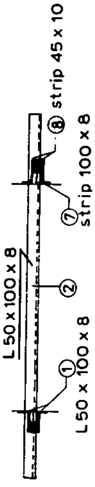


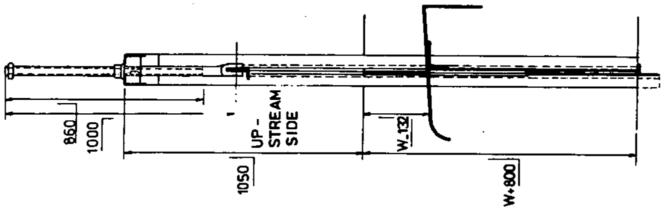
WEIR TABLE AND HOISTING BEAM

TERMINAL GATES



UPSTREAM VIEW





SECTION A_B

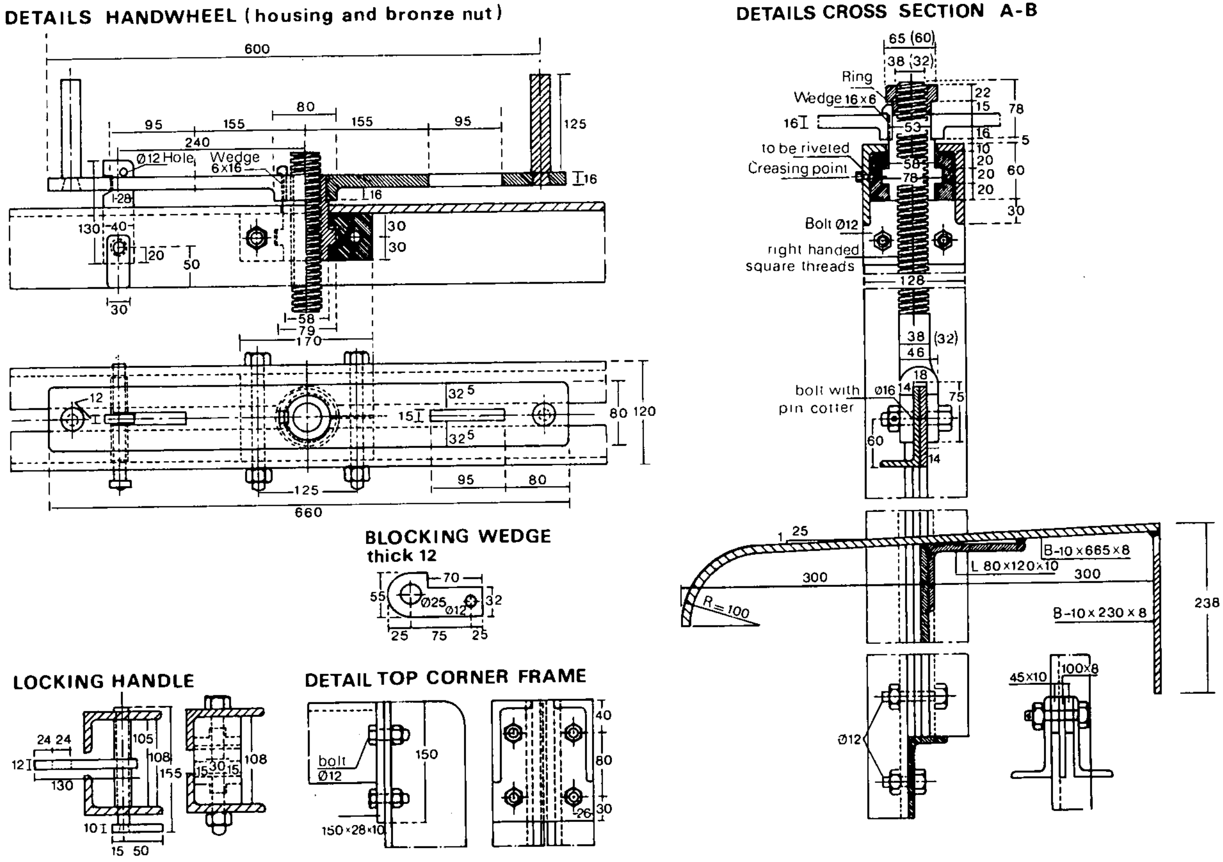


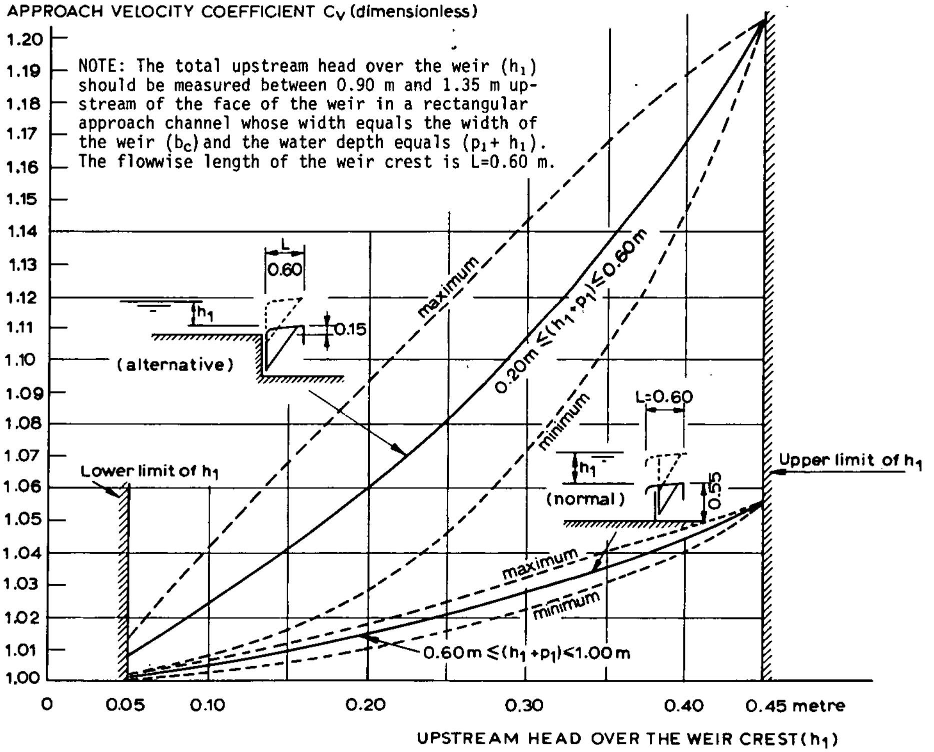
Figure 4.7 The Romijn movable measuring/regulating weir (dimensions in mm)

If the Romijn weir is installed in accordance with Figure 4.3, which is the normal method of installation, the values for h_1 and p_1 vary in such a way that

$$\begin{aligned} 0.05\text{ m} &\leqslant h_1 &&\leqslant 0.45\text{ m} \\ 0.55\text{ m} &\leqslant p_1 &&\leqslant 0.95\text{ m} \\ 0.60\text{ m} &\leqslant h_1 + p_1 &&\leqslant 1.00\text{ m} \end{aligned}$$

Due to the variation of both h_1 and p_1, the approach velocity coefficient is not a function of h_1 alone, but ranges between the broken lines shown in Figure 4.8. In irrigation practice it is confusing to work with several C_v-values for the same upstream head. Therefore the use of an average C_v-value, as a function of the upstream head h_1 only, is advised. It follows from Figure 4.8 that this average C_v-value may be expected to have an error of less than 1%. The discharge in m^3/s per metre width of weir crest can be calculated from Equation 4-4 and Figures 4.5 and 4.8. Values of q for each 0.01 m of head are presented in Table 4.2, Column 2.

An alternative method of installing the weir is to use no bottom slide. The movable weir is then lowered behind a drop in the channel bottom, this drop acting as a bottom terminal. With this method, the height of the weir crest above the bottom of approach channel is less than with the normal method of installation. Consequently, the approach velocity and thus the C_v-value is significantly higher. For a standard weir with



NOTE: The total upstream head over the weir (h_1) should be measured between 0.90 m and 1.35 m upstream of the face of the weir in a rectangular approach channel whose width equals the width of the weir (b_c) and the water depth equals (p_1+ h_1). The flowwise length of the weir crest is L=0.60 m.

Figure 4.8 Approach velocity coefficient (C_v) as a function of the head over the movable weir crest (h_1)

a length of the weir crest in the direction of flow of 0.60 m, values of p_1 and h_1 range in such a way that:

$$\begin{array}{lll} 0.05\text{ m} \leqslant & h_1 & \leqslant 0.45\text{ m} \\ 0.15\text{ m} \leqslant & p_1 & \leqslant 0.55\text{ m} \\ 0.20\text{ m} \leqslant & h_1 + p_1 & \leqslant 0.60\text{ m} \end{array}$$

Values of the ratio $C_d h_1/(h_1 + p_1)$ thus range more widely than before, as do C_v values as a function of h_1. Minimum and maximum possible C_v-values are shown in Figure 4.8. Here, the average C_v-value to be used may be expected to have an error of less than 4%. Values of q for each 0.01 m of head may be calculated from Equation 4-4 and from Figures 4.5 and 4.8, and are presented in Table 4.2, Column 3.

4.2.5 Limits of application

The limits of application of a movable Romijn weir for reasonable accuracy are:

a. The practical lower limit of h_1 is related to fluid properties and to the accuracy with which gauge readings can be made. The recommended lower limit of h_1 is 0.05 m or 0.08 L, whichever is greater;
b. To reduce the influence of boundary layer effects at the sides of the weir, the weir breadth b_c should not be less than 0.30 m nor less than the maximum value of H_1;
c. The height of the weir crest above the bottom of the approach channel should not be less than 0.15 m nor less than 0.33 H_{1max};
d. To obtain a sensibly constant discharge coefficient, the ratio H_1/L should not exceed 0.75;
e. The submergence ratio H_2/H_1 should not exceed 0.30 to obtain modular flow.

4.3 Triangular broad-crested weir

4.3.1 Description

On natural streams where it is necessary to measure a wide range of discharges, a triangular control has several advantages. Firstly it provides a large breadth at high flows so that the backwater effect is not excessive. Secondly, at low flows the breadth is reduced so that the sensitivity of the weir remains acceptable. These advantages, combined with the fact that a triangular control section has a critical depth equal to 0.8 H_1 so that the weir can take a high submergence before its capacity is affected, makes this weir type an interesting flow measuring device. A description of the weir, although slightly different in shape, was published in 1964 by Bos.

The weir profile in the direction of flow shows an upstream rounded nose with a minimum radius r equal to 0.11 H_{1max} to prevent flow separation. For the economic design of field structures, however, a value $r = 0.20\ H_{1max}$ is recommended. To obtain a sensibly hydrostatic pressure distribution above the weir crest, the length of the horizontal portion of the crest should not be less than 1.75 H_{1max}. To obtain a favourable (high) discharge coefficient the crest length L should be close to the permissible minimum. The weir should be placed between vertical abutments and be at right angles

Table 4.2 Discharge per metre width of weir crest for the movable Romijn measuring/regulating weir

Head h_1 metre	Discharge q in m^3/s per metre width for two methods of installation	
	normal $0.55\ m \leqslant p_1 \leqslant 0.95\ m$	alternative $0.15\ m \leqslant p_1 \leqslant 0.55\ m$
0.05	0.0195	0.0196
0.06	.0258	.0260
0.07	.0327	.0332
0.08	.0402	.0408
0.09	.0483	.0491
0.10	.0568	.0579
0.11	.0658	.0672
0.12	.0752	.0770
0.13	.0850	.0873
0.14	.0952	.0980
0.15	.106	.109
0.16	.117	.121
0.17	.128	.133
0.18	.140	.145
0.19	.152	.158
0.20	.164	.171
0.21	.176	.185
0.22	.189	.199
0.23	.202	.213
0.24	.216	.228
0.25	.230	.243
0.26	.244	.259
0.27	.258	.275
0.28	.273	.292
0.29	.288	.310
0.30	.304	.327
0.31	.319	.345
0.32	.336	.365
0.33	.353	.384
0.34	.370	.404
0.35	.388	.426
0.36	.407	.448
0.37	.425	.470
0.38	.444	.493
0.39	.464	.517
0.40	.484	.541
0.41	.504	.566
0.42	.525	.592
0.43	.547	.619
0.44	.569	.646
0.45	.591	.675

NOTE: The number of corresponding figures given in the columns for discharge should not be taken to imply a corresponding accuracy of the values given, but only to assist in the interpolation and rounding off for various values of head.

to the direction of flow. The upstream head over the weir crest should be measured in the rectangular approach channel at a distance of between two and three times H_{1max} upstream from the weir face (see also Chapter 2).

Essentially, there are two types of triangular broad-crested weirs:

(i) if the maximum weir width is unrestricted (i.e. if the available weir width is such that in combination with a selected weir notch angle θ, the water level in the control section does not reach the intersection of side slopes and vertical abutments), the weir type is referred to as 'less-than-full'. For this type of weir, one head-discharge equation applies for the entire operating range from H_{1min} to H_{1max}.

(ii) if the weir is installed in a channel with restricted width, the water level at the control section may sometimes rise above the top of the side slopes. This weir type is referred to as 'over-full', and somewhere in between H_{1min} and H_{1max} we have to change over from the head-discharge equation for a triangular control section to that of a truncated triangular control section.

As shown in Sections 1.9.3 and 1.9.4, critical depth in a triangular control section

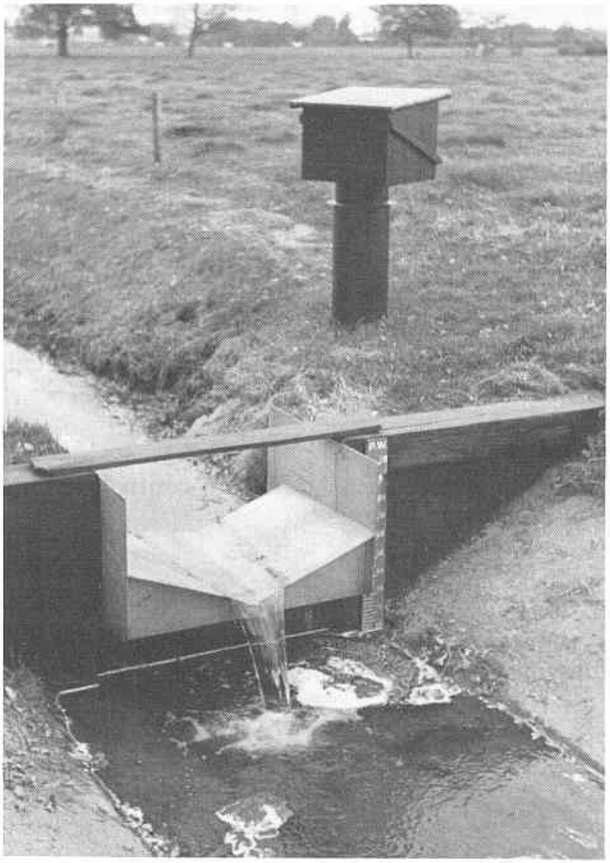


Photo 2 Triangular broad-crested weir

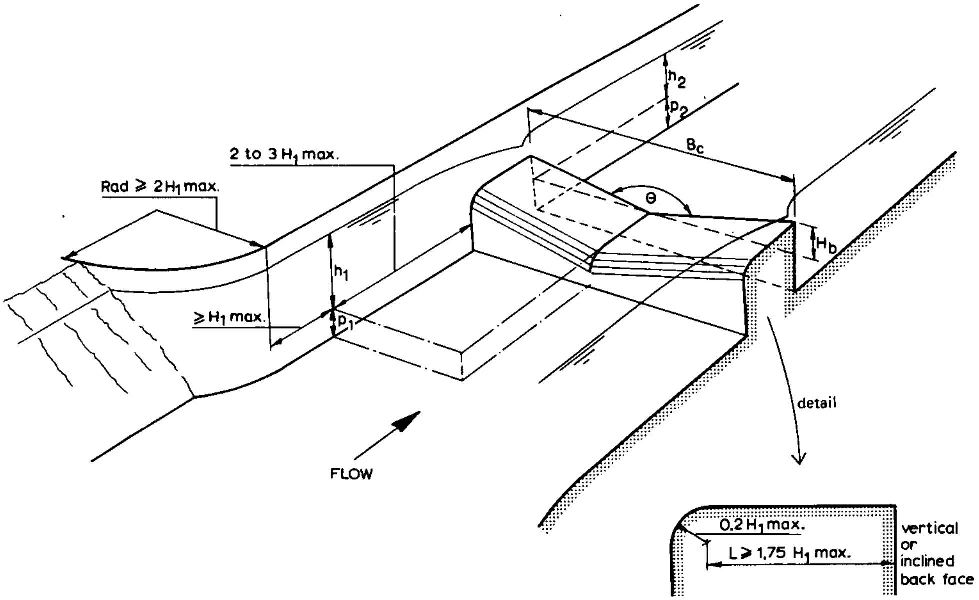


Figure 4.9 Definition sketch for triangular broad-crested weir

equals $y_c = 0.80\ H_1$, so that the weir is just full if $H_b = 0.80\ H_1$ or $H_1 = 1.25\ H_b$, where H_b denotes the difference in elevation between the top of the side slopes and the vertex of the weir notch (see Figure 4.9) and equals $H_b = {}^1/_2\ B_c \cot \theta/2$.

4.3.2 Evaluation of discharge

As discussed already in Section 4.3.1 we can distinguish between two different cases of head-discharge relationships, as follows

'Less-Than-Full' ($H_1 \leqq 1.25\ H_b$)

In this case the basic head-discharge equation for a triangular control section is applicable, which, according to Section 1.9.3, reads

$$Q = C_d C_v \frac{16}{25}\left[\frac{2}{5}g\right]^{0.50} \tan\frac{\theta}{2}\, h_1^{2.50} \qquad (4\text{-}5)$$

where the discharge coefficient may be read as a function of the ratio H_1/L from Figure 4.10. The approach velocity coefficient may be read from Figure 1.12 as a function of the dimensionless ratio

$$C_d \frac{A^*}{A_1} = C_d \times \frac{h_1^2 \tan\theta/2}{B_c(h_1 + p_1)}$$

'Over-Full' ($H_1 \geqslant 1.25\ H_b$)

In this case the basic head-discharge equation for a truncated triangular control section applies (see Section 1.9.4)

$$Q = C_d C_v \frac{2}{3}\left[\frac{2}{3} g\right]^{0.50} B_c(h_1 - {}^1\!/_2 H_b)^{1.50} \qquad (4\text{-}6)$$

where values of C_d again may be read from Figure 4.10 as a function of the ratio H_1/L. It should be noted that if H_1/L exceeds 0.50 the weir cannot be termed broad-crested. If ratios $H_1/L \geqslant 0.50$ are used, the overfalling nappe should be fully aerated, and it should be noted that the modular limits given in Section 4.3.2 will decrease significantly with increasing H_1/L-values. C_v-values may be obtained from Figure 1.12 as a function of the dimensionless ratio $C_dA^*/A_1 = C_d(h_1 - {}^1\!/_2 H_b)/(h_1 + p_1)$.

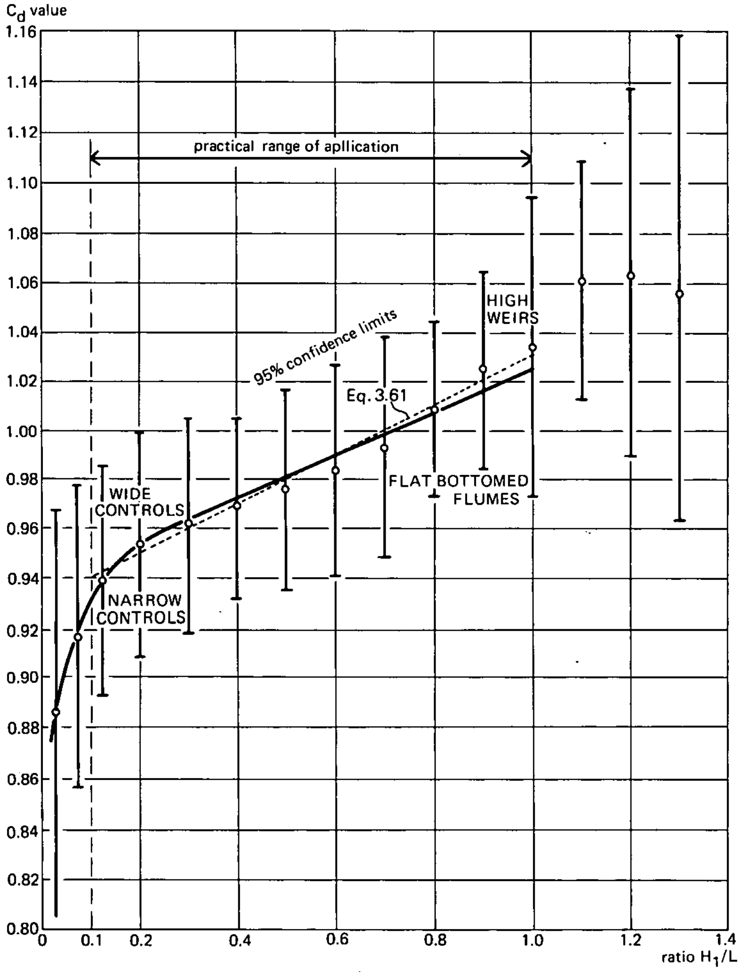


Figure 4.10 C_d values as a function of H_1/L of broad-crested weirs and long-throated flumes of all shapes and sizes (Bos 1985)