Annex 4
Suitable stilling basins

4.1 Introduction

Unless a weir or flume is founded on rock, a downstream stilling basin will be necessary. The floor of the stilling basin should be set at such a level that the hydraulic jump, if formed, occurs on the sloping downstream weir face or at the upstream end of the basin floor so that the turbulence in the jump will abate to a level which will not damage the unprotected downstream channel bed. Calculations for the floor level should be made for several discharges throughout the anticipated range of modular flow. To aid the engineer in designing a suitable stilling basin, hydraulic design criteria of a number of devices are given below.

4.2 Straight drop structures
4.2.1 Common drop

Illustrated in Figure A4.1 is a drop structure that will dissipate energy if installed downstream of a weir with a vertical back face. The aerated free falling nappe will strike the basin floor and turn downstream at Section U. Beneath the nappe a pool is formed which supplies the horizontal thrust required to turn the nappe downstream. Because of the impact of the nappe on the basin floor and the turbulent circulation in the pool beneath the nappe, some energy is lost.

Further energy will be dissipated in the hydraulic jump downstream of section U. The remaining energy head downstream from the basin, H_d, does not vary greatly

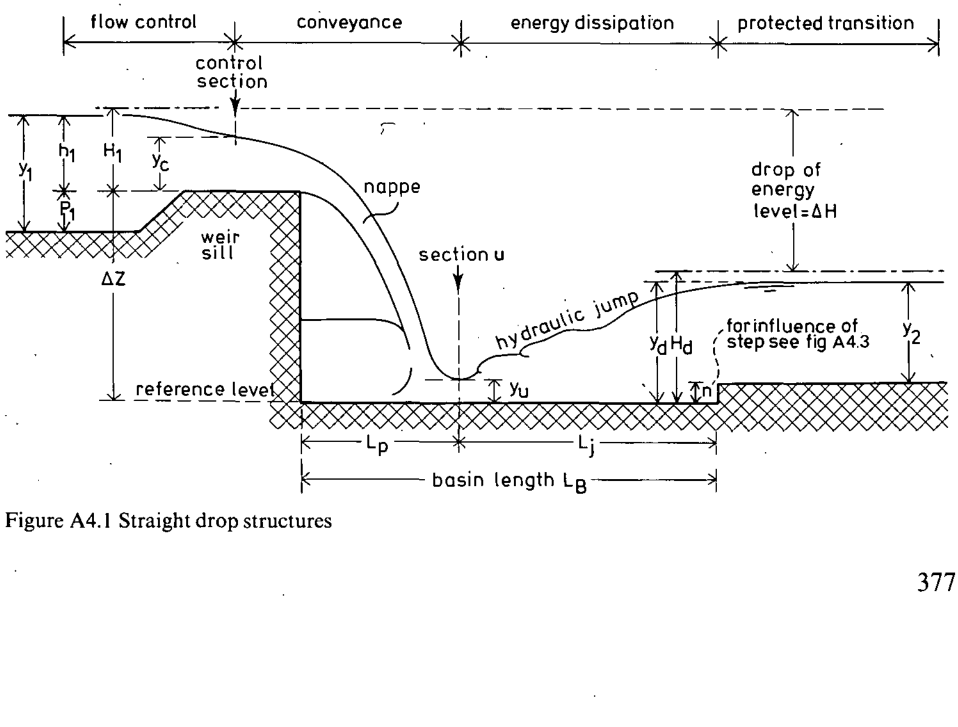


Figure A4.1 Straight drop structures

with the ratio $\Delta Z/H_1$ and is equal to about 1.67 H_1 (adapted from Henderson 1966). This value of 1.67 H_1 provides a satisfactory estimate for the basin floor level below the energy level of the downstream canal. The hydraulic dimensions of a straight drop can be related to the following variables (see Figure A4.1):

H_1 = upstream sill-referenced energy head
ΔH = change in energy head across structure
H_d = downstream energy head
q = discharge per unit width of sill
g = accelaration due to gravity
n = step height
y_u = flow depth at section U
y_d = downstream flow depth relative to basin floor
y_2 = flow depth in downstream channel

These variables can be combined to make a first estimate of the drop height

$$\Delta Z = (\Delta H + H_d) - H_1 \tag{A4.1}$$

Subsequently, the flow velocity and depth at section U may be estimated by

$$v_u = \sqrt{2g\Delta Z} \tag{A4.2}$$

and by the continuity equation

$$y_u = \frac{q}{v_u} \tag{A4.3}$$

The flow at section U can best be characterized by the dimensionless Froude number

$$Fr_u = \frac{v_u}{\sqrt{gy_u}} \tag{A4.4}$$

This Froude number can be related directly to the straight drop geometry through the length ratios $y_d/\Delta Z$ and $L_p/\Delta H$, values of which can be read from Figure A4.2 (see also Figure A4.1).

The length of the hydraulic jump L_j, downstream from section U in Figure A4.1, can be calculated by (Henderson 1966),

$$L_j = 6.9\,(y_d - y_u) \tag{A4.5}$$

It is important to realize that the downstream water depths (y_d and y_2) are caused not by the drop structure, but by the flow characteristics of the downstream canal. If these characteristics are such that the required depth y_d is produced, a jump will form; otherwise it will not form and not enough energy will be dissipated within the basin. Additional steps, such as lowering the basin floor and adding an end sill, must be taken to assure adequate energy dissipation.

Because of seasonal changes of the hydraulic resistance of the canal, however, the flow velocity as calculated by Manning's equation changes together with the water depth y_d. The jump thus tends to drift up and down the canal. This unstable behavior is often undesirable, and is then suppressed by increasing the flow resistance by means of an abrupt step at the end of the basin. Usually, this step is constructed at a distance

$$L_j = 5(n + y_2) \tag{A4.6}$$

downstream of section U. For design purposes, Figure A4.3 can be used to determine the largest required value of n, if $Fr_u = v_u/\sqrt{gy_u}$, y_u, and y_2 are known.

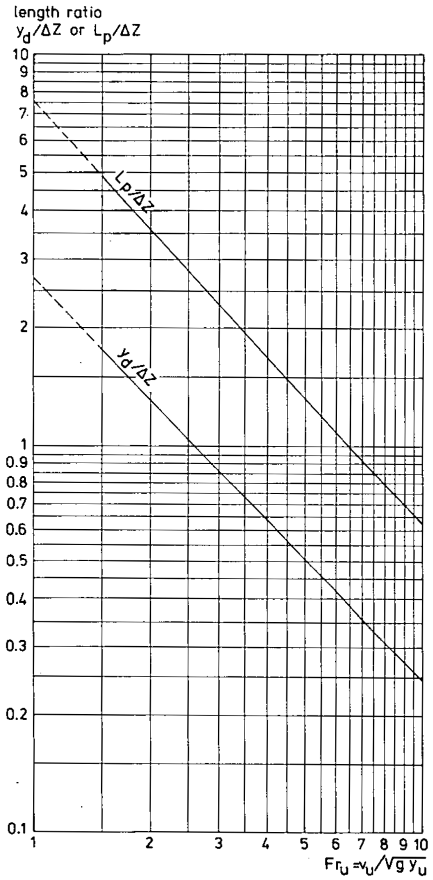


Figure A4.2 Dimensionless plot of straight drop geometry (Bos e.a. 1984)

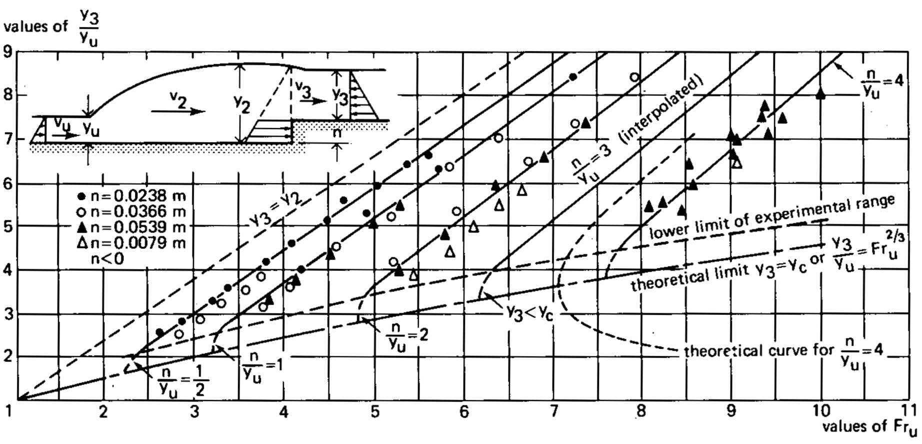


Figure A4.3 Experimental relationships between Fr_u, y_2/y_u, and n/y_u for an abrupt step (after Forster and Skrinde 1950)

4.2.2 U.S. ARS basin

The U.S. Agricultural Research Service has developed an alternative basin which is especially suitable if tailwater level is greater than the sequent depth and varies independently of the flow rate. This impact block type basin was developed for low heads and gives a good energy dissipation over a wide range of tailwater levels. The energy dissipation is principally by turbulence induced by the impingement of the incoming jet upon the impact blocks. The required downstream water depth, therefore, can be slightly less than with the previous basin but can vary independently of the drop height ΔZ. To function properly, the downstream water depth y_d must not be less than $1.45\ H_1$, while at Q_{max} the Froude number Fr_u should not exceed 4.5.

Upstream from section U, the length L_p may be determined by use of Figure A4.2. The linear dimensions of the basin downstream from section U are shown in Figure A4.4 as a function of H_1.

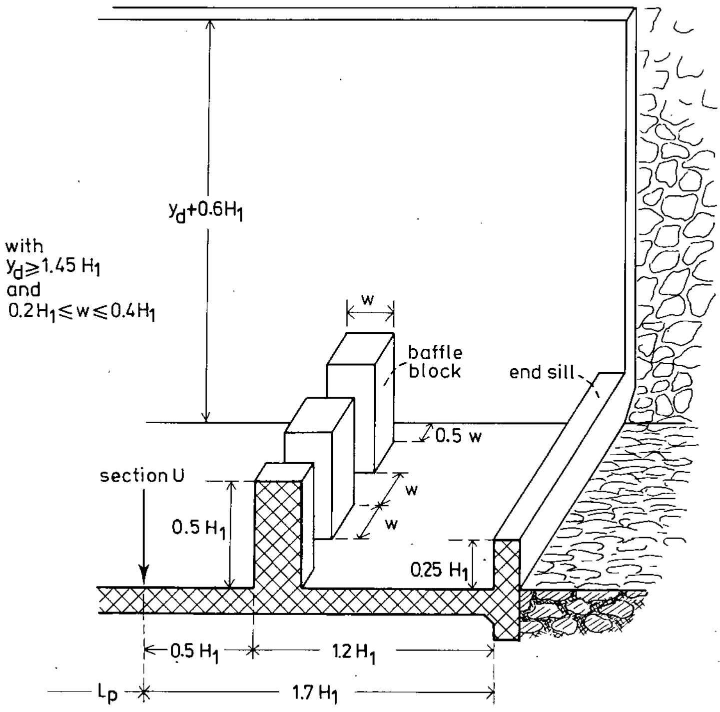


Figure A4.4 Impact block type basin

4.3 Inclined drops or chutes

4.3.1 Common chute

Downstream from the control section of either a weir or flume, a sloping downstream face or expansion is a common design feature. The slope of the downstream face usually varies between 1 to 4 and 1 to 6. By approximation we may write that the flow velocity over the downstream face equals

$$v_u = q/y_u \tag{A4.7}$$

where q is the unit discharge on the downstream face and y_u is the water depth at a particular point on the downstream apron.

Values of y_u may be determined by the use of Table A4.1. The symbols used in Table A4.1 are defined in Figure A4.5.

A hydraulic jump will form in the horizontal (rectangular) basin provided that the tailwater depth is greater than the sequent depth y_2 to y_u and v_u. Minimum values of y_2 may be read from Figure A4.3 for rectangular basins. The length of such a horizontal basin equals that part of the basin which is situated downstream of Section U in Figure A4.1, and equals $L_j = 5(n + y_2)$.

It is recommended that a tabulation be made of the Froude number Fr_u near the toe of the downstream face, and of the depth of flow y_u throughout the anticipated

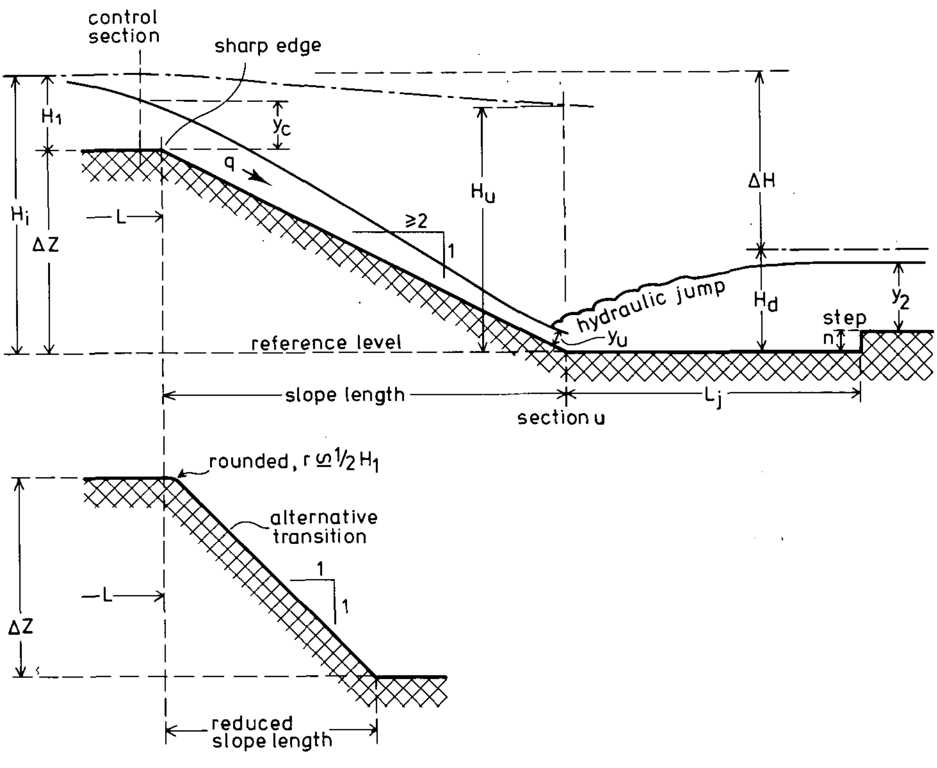


Figure A4.5 Definition sketch for Table A4.1

Table A4.1 Dimensionless Ratios for Hydraulic Jumps

$\frac{\Delta H}{H_1}$	$\frac{y_d}{y_u}$	$\frac{y_u}{H_1}$	$\frac{v_u^2}{2gH_1}$	$\frac{H_u}{H_1}$	$\frac{y_d}{H_1}$	$\frac{v_d^2}{2gH_1}$	$\frac{H_d}{H_1}$
0.2446	3.00	0.3669	1.1006	1.4675	1.1006	0.1223	1.2229
0.2688	3.10	0.3599	1.1436	1.5035	1.1157	0.1190	1.2347
0.2939	3.20	0.3533	1.1870	1.5403	1.1305	0.1159	1.2464
0.3198	3.30	0.3469	1.2308	1.5777	1.1449	0.1130	1.2579
0.3465	3.40	0.3409	1.2749	1.6158	1.1590	0.1103	1.2693
0.3740	3.50	0.3351	1.3194	1.6545	1.1728	0.1077	1.2805
0.4022	3.60	0.3295	1.3643	1.6938	1.1863	0.1053	1.2916
0.4312	3.70	0.3242	1.4095	1.7337	1.1995	0.1030	1.3025
0.4609	3.80	0.3191	1.4551	1.7742	1.2125	0.1008	1.3133
0.4912	3.90	0.3142	1.5009	1.8151	1.2253	0.0987	1.3239
0.5222	4.00	0.3094	1.5472	1.8566	1.2378	0.0967	1.3345
0.5861	4.20	0.3005	1.6407	1.9412	1.2621	0.0930	1.3551
0.6525	4.40	0.2922	1.7355	2.0276	1.2855	0.0896	1.3752
0.7211	4.60	0.2844	1.8315	2.1159	1.3083	0.0866	1.3948
0.7920	4.80	0.2771	1.9289	2.2060	1.3303	0.0837	1.4140
0.8651	5.00	0.2703	2.0274	2.2977	1.3516	0.0811	1.4327
0.9400	5.20	0.2639	2.1271	2.3910	1.3723	0.0787	1.4510
1.0169	5.40	0.2579	2.2279	2.4858	1.3925	0.0764	1.4689
1.0957	5.60	0.2521	2.3299	2.5821	1.4121	0.0743	1.4864
1.1763	5.80	0.2467	2.4331	2.6798	1.4312	0.0723	1.5035
1.2585	6.00	0.2417	2.5372	2.7789	1.4499	0.0705	1.5203
1.3429	6.20	0.2367	2.6429	2.8796	1.4679	0.0687	1.5367
1.4280	6.40	0.2321	2.7488	2.9809	1.4858	0.0671	1.5529
1.5150	6.60	0.2277	2.8560	3.0837	1.5032	0.0655	1.5687
1.6035	6.80	0.2235	2.9643	3.1878	1.5202	0 0641	1.5843
1.6937	7.00	0.2195	3.0737	3.2932	1.5368	0.0627	1.5995
1.7851	7.20	0.2157	3.1839	3.3996	1.5531	0.0614	1.6145
1.8778	7.40	0.2121	3.2950	3.5071	1.5691	0.0602	1.6293
1.9720	7.60	0.2085	3.4072	3.6157	1.5847	0.0590	1.6437
2.0674	7.80	0.2051	3.4723	3.7254	1.6001	0.0579	1.6580
2.1641	8.00	0.2019	3.6343	3.8361	1.6152	0.0568	1.6720
2.2620	8.20	0.1988	3.7490	3.9478	1.6301	0.0557	1.6858
2.3613	8.40	0.1958	3.8649	4.0607	1.6446	0.0548	1.6994
2.4615	8.60	0.1929	3.9814	4.1743	1.6589	0.0538	1.7127
2.5630	8.80	0.1901	4.0988	4.2889	1.6730	0.0529	1.7259
2.6656	9.00	0.1874	4.2171	4.4045	1.6869	0.0521	1.7389
2.7694	9.20	0.1849	4.3363	4.5211	1.7005	0.0512	1.7517
2.8741	9.40	0.1823	4.4561	4.6385	1.7139	0.0504	1.7643
2.9801	9.60	0.1799	4.5770	4.7569	1.7271	0.0497	1.7768
3.0869	9.80	0.1775	4.6985	4.8760	1.7402	0.0489	1.7891
3.1949	10.00	0.1753	4.8208	4.9961	1.7530	0.0482	1.8012
3.4691	10.50	0.1699	5.1300	5.2999	1.7843	0.0465	1.8309
3.7491	11.00	0.1649	5.4437	5.6087	1.8146	0.0450	1.8594
4.0351	11.50	0.1603	5.7623	5.9227	1.8439	0.0436	1.8875
4.3267	12.00	0.1560	6.0853	6.2413	1.8723	0.0423	1.9146
4.6233	12.50	0.1520	6.4124	6.5644	1.9000	0.0411	1.9411
4.9252	13.00	0.1482	6.7437	6.8919	1.9268	0.0399	1.9667
5.2323	13.50	0.1447	7.0794	7.2241	1.9529	0.0389	1.9917
5.5424	14.00	0.1413	7.4189	7.5602	1.9799	0.0379	2.0178
5.8605	14.50	0.1381	7.7625	7.9006	2.0032	0.0369	2.0401
6.1813	15.00	0.1351	8.1096	8.2447	2.0274	0.0361	2.0635
6.5066	15.50	0.1323	8.4605	8.5929	2.0511	0.0352	2.0863
6.8363	16.00	0.1297	8.8153	8.9450	2.0742	0.0345	2.1087
7.1702	16.50	0.1271	9.1736	9.3007	2.0968	0.0337	2.1305
7.5081	17.00	0.1247	9.5354	9.6601	2.1190	0.0330	2.1520
7.8498	17.50	0.1223	9.9005	10.0229	2.1407	0.0323	2.1731
8.1958	18.00	0.1201	10.2693	10.3894	2.1619	0.0317	2.1936
8.5438	18.50	0.1180	10.6395	10.7575	2.1830	0.0311	2.2141
8.8985	19.00	0.1159	11.0164	11.1290	2.2033	0.0305	2.2339
9.2557	19.50	0.1140	11.3951	11.5091	2.2234	0.0300	2.2534
9.6160	20.00	0.1122	11.7765	11.8887	2.2432	0.0295	2.2727

discharge range. The sequent depth rating should be plotted with the stage-discharge curve of the tailwater channel to ensure that the jump forms on the basin floor.

4.3.2 SAF Basin

An alternative stilling basin suitable for use on low-head structures was developed at the St. Anthony Falls Hydraulic Laboratory (SAF-basin) of the University of Minnesota. The basin is used as a standard by the U.S. Soil Conservation Service, and has been reported by Blaisdell (1943, 1959). The general dimensions of the SAF-basin are shown in Figure A4.6.

The design parameters for the SAF-basin are given in Table A4.2.

Table A4.2 Design parameters of the SAF-basin

$Fr_u = v_u \sqrt{gA_u/B_u}$	L_B/y_2	TW/y_2
1.7 to 5.5	$4.5/Fr_u^{0.76}$	$1.1 - Fr_u^2/120$
5.5 to 11	$4.5/Fr_u^{0.76}$	0.85
11 to 17	$4.5\, Fr_u^{0.76}$	$1.0 - Fr_u^2/800$

In Table A4.2 y_2 is the theoretical sequent depth of the jump corresponding to y_u as shown in Figure A4.3. The height of the end sill is given by $C = 0.07\ y_2$ and the freeboard of the sidewall above the maximum tailwater depth to be expected during the life of the basin is given by $z = y_2/3$.

The sidewalls of the basin may be parallel or they may diverge. Care should be taken that the floor blocks occupy between 40 and 55% of the stilling basin width, so that their width and spacing must be increased with the amount of divergence of the sidewalls. The effect of air entrainment should not be taken into account in the design of the basin; however, its existence within the stilling basin calls for a generous freeboard ($y_2/3$).

4.4 Riprap protection

To prevent bank damage by erosive currents passing over the end sill of a basin or leaving the tail of a structure, riprap is usually placed on the downstream channel bottom and banks. Several factors affect the stone size required to resist the forces which tend to move riprap. In terms of flow leaving a structure, these factors are velocity, flow direction, turbulence and waves. The purpose of this section is to give the design engineer a tool to determine the size of riprap to be used downstream from discharge measurement devices or stilling basins and to determine the type of filter or bedding material placed below the riprap.

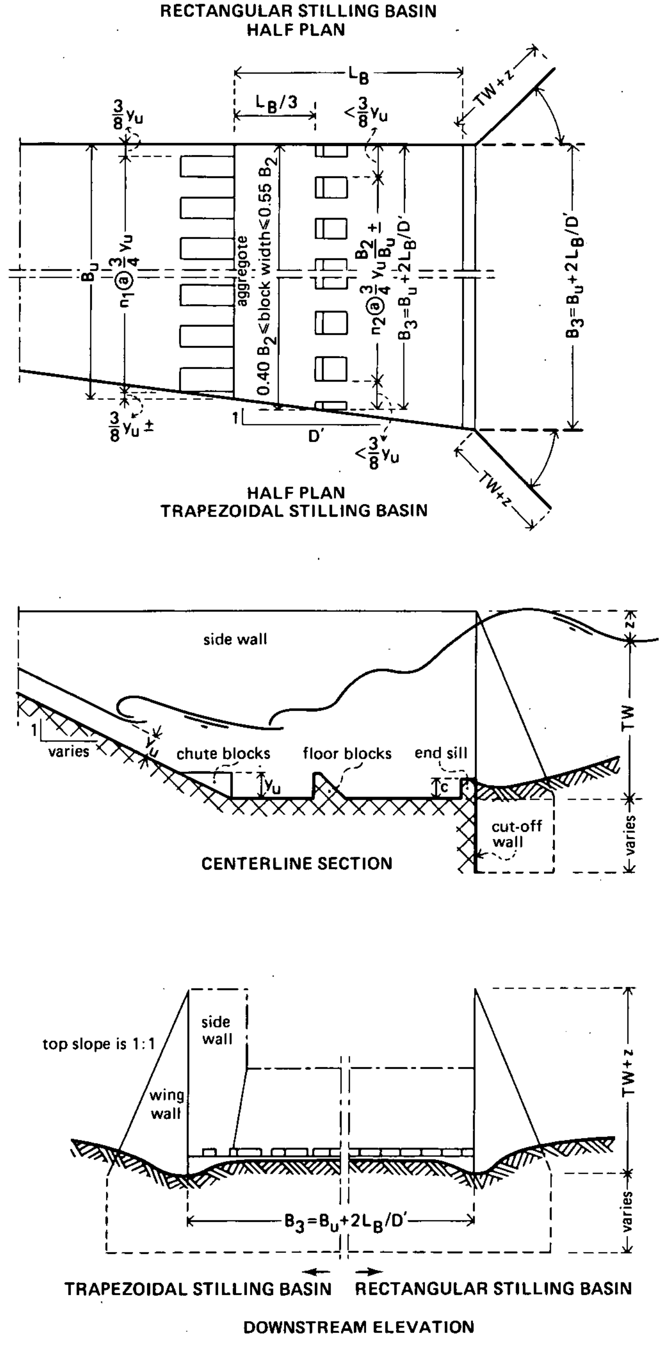
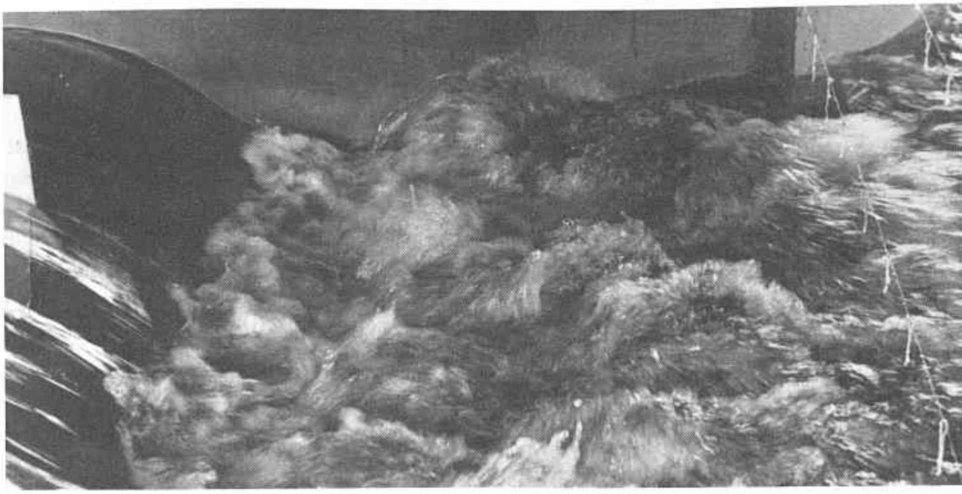


Figure A4.6 SAF-basin dimensions



Design A Tailwater depth calculated by $TW/y_2 = 1.1 - Fr_u^2/120$

Design B Tailwater depth is 15% greater than in Design A

PHOTOS: 1:20 scale model of SAF stilling basin discharging 1200 m^3/s in prototype
$b_c = 40.0$ m, $\Delta H = 3.50$ m

4.4.1 Determining maximum stone size in riprap mixture

From published data, a tentative curve was selected showing the minimum stone diameter as a function of the bottom velocity. This curve is shown in Figure A4.7. Downstream of stilling basins, the conception 'bottom velocity' is difficult to define because of the highly turbulent flow pattern. The velocity at which the water strikes the riprap is rather unpredictable unless the basin is tested.

For practical purposes, however, Peterka (1964) recommends that, to find the stone diameter in Figure A4.7, use be made of the average velocity based on discharge divided by cross-sectional area at the end sill of the stilling basin. If no stilling basin is needed because $Fr_u < 1.7$, Figure A4.7 should be entered with the impact velocity, being

$$v_u = \sqrt{2g\,\Delta Z} \tag{A4.8}$$

More than 60% of the riprap mixture should consist of stones which have length, width, and thickness dimensions as nearly alike as is practicable, and be of curve size or larger; or the stones should be of curve weight or heavier and should not be flat slabs.

4.4.2 Filter material placed beneath riprap

If riprap stones of a protective lining were to be installed directly on top of the fine material in which the canal is excavated, grains of this subgrade would be washed through the openings in between the riprap stones. This process is partly due to the turbulent flow of canal water in and out of the voids between the stones and partly due to the inflow of water that leaks around the structure or flows into the drain.

To avoid damage to a riprap protection because of the washing of subgrade, a filter must be placed between the riprap and the subgrade (see Figure A4.8). The protective construction as a whole and each separate layer must be sufficiently permeable to water entering the canal through its bed or banks. Further, fine material from an underlying filter layer or the subgrade must not be washed into the voids of a covering layer.

4.4.3 Permeability to water

To maintain a sufficient permeability to water of the protective construction of Figure A4.8, the following d_{15}/d_{15} ratios should have a value between 5 and 40 (USBR 1973):

$$\frac{d_{15}\text{ layer 3}}{d_{15}\text{ layer 2}} \text{ and } \frac{d_{15}\text{ layer 2}}{d_{15}\text{ layer 1}} \text{ and } \frac{d_{15}\text{ layer 1}}{d_{15}\text{ subgrade}} = 5 \text{ to } 40 \tag{A4.9}$$

where d_{15} equals the diameter of the sieve opening whereby 15% of the total weight of the sample passes the sieve. Depending on the shape and gradation of the grains in each layer, the above-mentioned 5 to 40 range of the ratios can be narrowed as follows (Van Bendegom 1969):

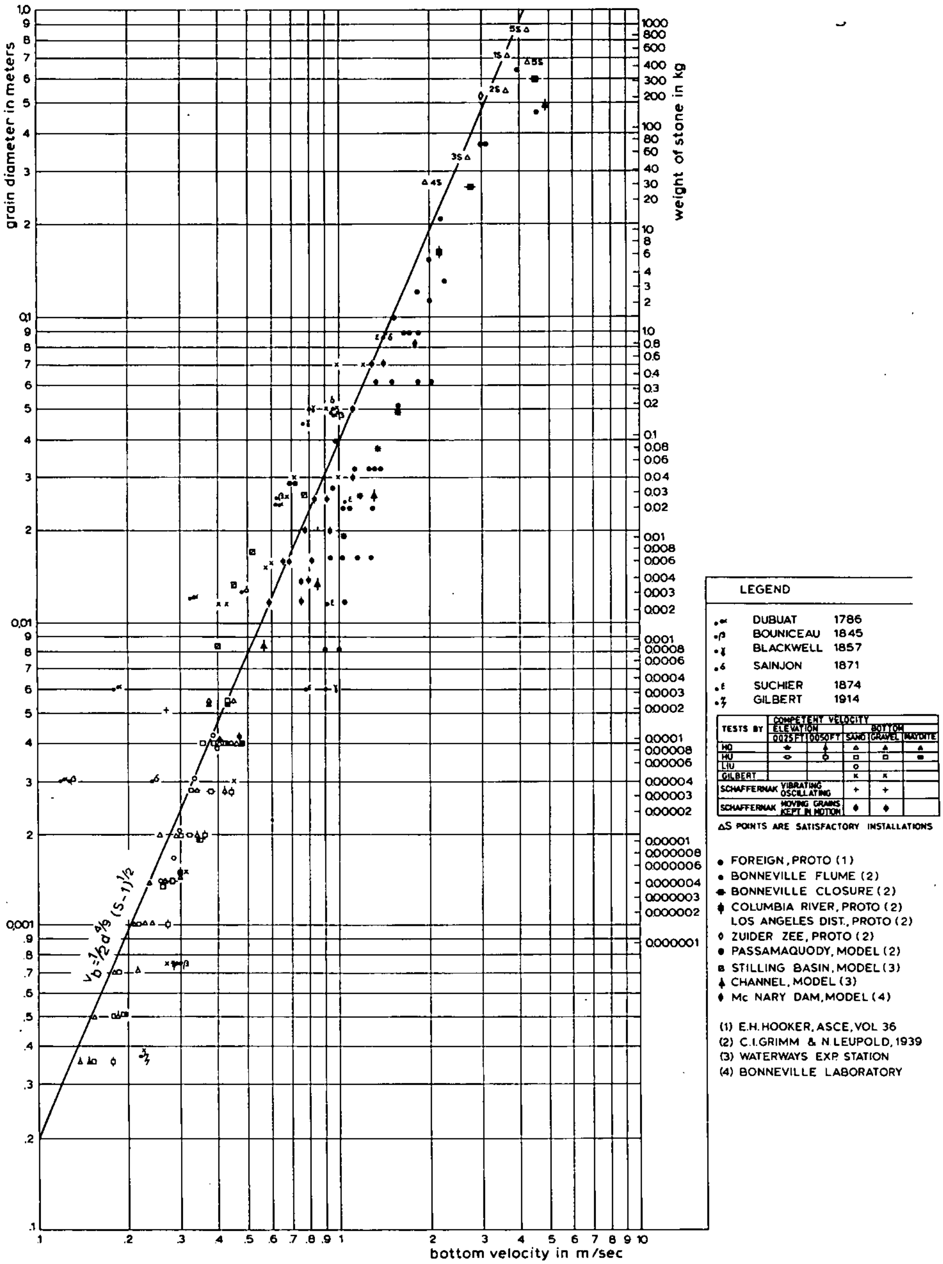


Figure A4.7 Curve to determine maximum stone size

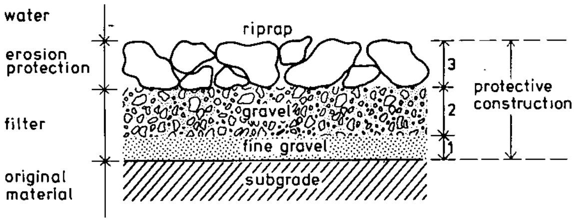


Figure A4.8 Example of filter between riprap and original material (subgrade) in which canal is excavated

1.	Homogeneous round grains (gravel)	5 to 10
2.	Homogeneous angular grains (broken gravel, rubble)	6 to 20
3.	Well-graded grains	12 to 40

To prevent the filter from clogging it is, in addition, advisable that for each layer

$$d_5 \geqslant 0.75 \text{ mm} \tag{A4.10}$$

4.4.4 Stability of each layer

To prevent the loss of fine material from an underlying filter layer or the subgrade through the openings in a covering layer, two requirements must be met:

The following d_{15}/d_{85} ratios should not exceed 5 (Bertram 1940)

$$\frac{d_{15} \text{ layer 3}}{d_{85} \text{ layer 2}} \text{ and } \frac{d_{15} \text{ layer 2}}{d_{85} \text{ layer 1}} \text{ and } \frac{d_{15} \text{ layer 1}}{d_{85} \text{ subgrade}} \leqslant 5 \tag{A4.11}$$

while the d_{50}/d_{50} should range between 5 and 60 (U.S. Army Corps of Engineers 1955).

$$\frac{d_{50} \text{ layer 3}}{d_{50} \text{ layer 2}} \text{ and } \frac{d_{50} \text{ layer 2}}{d_{50} \text{ layer 1}} \text{ and } \frac{d_{50} \text{ layer 1}}{d_{50} \text{ subgrade}} = 5 \text{ to } 60 \tag{A4.12}$$

As before, the ratio in Equation A4.12 depends on the shape and graduation of the grains as follows:

1.	Homogeneous round grains (gravel)	5 to 10
2.	Homogeneous angular grains (broken gravel, rubble)	10 to 30
3.	Well-graded grains	12 to 60

The requirements in this section describe the sieve curves of the successive filter layers. Provided that the sieve curve of the riprap layer and the subgrade are known, other layers can be plotted. An example of plotting sieve curves of a construction consisting of one riprap and two filter layers is shown in Figure A4.9. In practice one should use materials that have a grain size distribution which is locally available, since it is uneconomic to compose a special mixture. To provide a stable and effectively functioning filter, the sieve curves for subgrade and filter layers should run about parallel for the small-diameter grains.

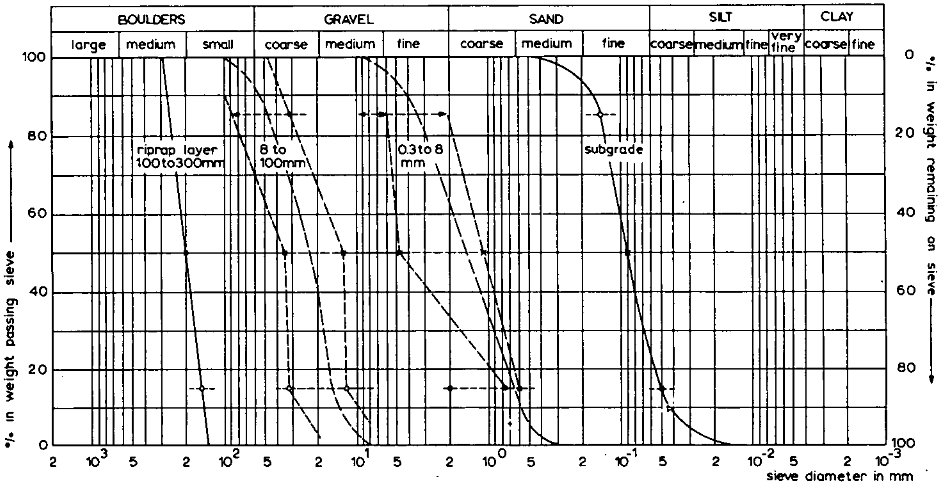


Figure A4.9 Sieve curves of a filter construction

4.4.5 Filter construction

To obtain a fair grain size distribution throughout a filter layer, each layer should be sufficiently thick. The following thicknesses must be regarded as a minimum for a filter construction made in the dry

- sand, fine gravel 0.05 to 0.10 m
- gravel 0.10 to 0.20 m
- stones 1.5 to 2 times the largest stone diameter.

With filters constructed under water, these thicknesses have to be increased considerably to compensate for irregularities in the subgrade and because it is more difficult to apply an even layer under water.

Many variations can be made on the basic filter construction. One or more of the layers can be replaced with other materials. With some protective linings, only the riprap layer is maintained, while the underlying layers are replaced by one single layer. For example

- concrete blocks on a nylon filter
- stones on braided azobe slabs on plastic filter
- gabions on fine gravel
- nylon-sand mattresses

The usual difficulty with these variants is their perviousness to underlying sand. The openings in each layer should not be greater than 0.5 × d_{85} of the underlying material. If openings are greater, one should not replace all underlying layers but maintain as many layers (usually one) as are needed to prevent the subgrade from being washed through the combined layer.

At structure-to-filter and filter-to-unprotected channel 'joints', the protective construction is most subject to damage. This is because the filter layer is subject to subsi-

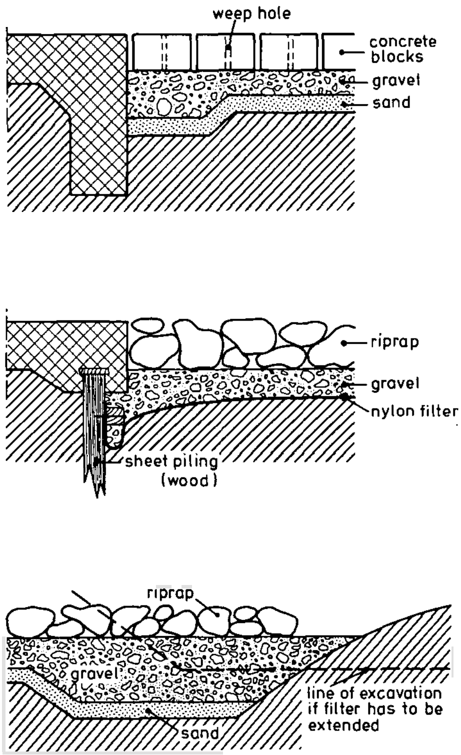


Figure A4.10 Examples of filter construction details (after van Bendegom 1969)

dence while the (concrete) structure itself is well founded. Underlying material (subgrade) may be washed out at these joints if no special measures are taken. It is recommended that the thickness of the filter construction be increased at these places. Some examples of common constructional details are shown in Figure A4.10.

As a rule of thumb we may suggest a length of riprap protection which is neither less than 4 times the (maximum) normal depth in the tailwater channel, nor less than the length of the earth transition, nor less than 1.50 m.

4.5 Selected list of references

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List of principal symbols

A	cross-sectional area	L^2
a	height of rectangular weir section (Sutro)	L
a	acceleration	LT^{-2}
B	channel surface width	L
b_c	breadth at bottom of control section	L
b_e	effective breadth of weir crest ($b_c + K_b$)	L
C_d	discharge coefficient	dimensionless
C_v	approach velocity coefficient	dimensionless
C_e	effective discharge coefficient (C_dC_v)	dimensionless
c	subscript for critical flow condition	dimensionless
D	diameter of float	L
D_p	diameter of pipe	L
d_c	diameter of circular weir	L
E	energy	ML^2T^{-2}
E	complete elliptical integral of the first kind	dimensionless
e	exponential number, 2.71828	dimensionless
F	force	MLT^{-2}
F	coefficient correction factor	dimensionless
Fr	Froude number, $Q(B/gA^3)^{1/2}$	dimensionless
f	friction coefficient in the Darcy-Weisbach equation	dimensionless
f	drowned flow reduction factor	dimensionless
G	weight	MLT^{-2}
G	relative slope factor	dimensionless
g	gravitational acceleration	LT^{-2}
H	total energy head over crest	L
H_o	specific energy	L
H_1	total upstream energy head over crest	L
H_2	total downstream energy head over crest	L
h_1	upstream head over crest	L
h_2	tailwater head over crest	L
h_e	effective upstream head over crest ($h_1 + K_h$)	L
Δh	head loss over structure ($h_1 - h_2$)	L
K	weir constant	dimensionless
K	head loss coefficient	dimensionless
K	complete elliptical integral of the second kind	dimensionless
k	filling ratio circular weir $(h_1/d_c)^{0.5}$	dimensionless
k	acceleration due to mass forces	LT^{-2}
L	flowwise length of crest	L
L	length of channel reach	L
l	length of pipe	L
m	mass	M
m	coordinate direction (binormal)	dimensionless
n	coordinate direction (principal normal)	dimensionless
n	number of data	dimensionless

P	wetted perimeter of flow cross-section	dimensionless
P	pressure	$ML^{-1}T^{-2}$
p_1	height of crest above approach channel bed	L
p_2	height of crest above tailwater channel bed	L
Q	discharge rate	L^3T^{-1}
Q_r	discharge rate through rectangular section	L^3T^{-1}
Q_c	discharge rate through curved section	L^3T^{-1}
Q_{air}	volumetric air discharge rate	L^3T^{-1}
q	discharge per unit width	L^2T^{-1}
R	hydraulic radius (A/P)	L
R_b	radius of embankment	L
r	radius of circular weir	L
r	radius of curved streamline	L
r	radius of float-wheel	L
r	radius of round-nose weir crest	L
S	length of side weir	L
S_H	submergence ratio (H_2/H_1)	dimensionless
S_h	submergence ratio (h_2/h_1)	dimensionless
S_m	modular limit	dimensionless
s	coordinate direction (velocity direction)	dimensionless
T_f	resisting torque due to friction	ML^2T^{-2}
TW	tailwater level	L
t	time	T
u	power of head or of differential head	dimensionless
V	volume of fluid	L^3
v	fluid velocity	LT^{-1}
$\bar{v}$	average fluid velocity (Q/A)	LT^{-1}
W	friction force	MLT^{-2}
w	acceleration due to friction	LT^{-2}
w	underflow gate opening	L
X	relative error	dimensionless
X	horizontal distance	L
x	breadth of weir throat at height y above crest	L
x	factor due to boundary roughness	dimensionless
x	cartesian coordinate direction	dimensionless
Y	vertical distance	L
y	vertical depth of flow	L
y	coordinate direction	dimensionless
z	coordinate direction	dimensionless
z	side slope ratio horz/vert	dimensionless
ΔZ	drop height	L
α	velocity distribution coefficient	dimensionless
α	diversion angle	degrees
β	half angle of circular section ($^1/_2\ \alpha$)	degrees
γ	Q_{max}/Q_{min}	dimensionless
δ	error	dimensionless
δ	contraction coefficient	dimensionless

Δ	small increment of	dimensionless
Δ	$(\rho_s - \rho)/\rho$: relative density	dimensionless
θ	weir notch angle	degrees
θ	angle of circular section	degrees
π	circular circumference-diameter ratio; 3.1416	dimensionless
ρ	mass density of water	ML^{-3}
ρ_{air}	mass density of air	ML^{-3}
ρ_s	mass density of bed material	ML^{-3}
ω	circular section factor	dimensionless
ξ	friction loss coefficient	dimensionless
σ	standard deviation	dimensionless
σ'	relative standard deviation	dimensionless

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