USE OF GABIONS IN SMALL HYDRAULIC WORKS

SECTION 3

GABION STRUCTURE DESIGN OF DAM SPILLWAY

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The spillway is a very important component of any hydraulic work. The spillway crest's main function consists in fixing the maximum water level upstream the structure, and preventing water overflows. The spillway must be accurately dimensioned, so that it can evacuate the design flow calculated for a hydraulic work.

The main procedures used for designing the structures that generally compose a spillway (e.g. spillway channel, weirs) are briefly illustrated in this section, with reference to both hydraulic theory and stability computation procedures. These procedures are taken from elaborate theories, which, for the sake of simplicity, will be only briefly mentioned here. For an in-depth treatment of these theories, readers should refer to specific publications.

The problem of runoff's restitution to the original river bed

- ***energy dissipation***
- ***stilling basin***

In natural streams, the total hydraulic energy is uniformly dissipated along the streambed. However, if a small dam or weir is built, the ***energy dissipation*** on the dam's upstream side results substantially lower than it would be in natural conditions and the potential energy level is therefore high. When this high hydraulic energy is dissipated downstream the structure, it could cause serious scour problems in the streambed, unless the rise in energy created by the structure's installation is dissipated immediately beyond the structure. This can happen naturally, if the characteristics of the streambed in question allow it, or artificially, with the creation of a ***stilling basin***. Here, the water flowing throughout the spillway must lose a portion of its total energy so as to reach a lower energy level downstream, i.e. a level equal to the one it would have had in the absence of the dam or weir.

The energy dissipation that takes place as a result of the construction of a hydraulic structure can give rise to important erosion phenomena in the streambed. Locally, this will threaten the structure's stability. Downstream, it will scour the river's bed for a long reach. Therefore, avoiding the negative consequences of energy dissipation is one of the principal problems to be dealt with when designing a hydraulic work.

A common way to solve this problem consists in concentrating the energy in a circumscribed area, called stilling basin. For the importance of its function, this area should be carefully designed and realised.

3.1 – TYPE OF SPILLWAY

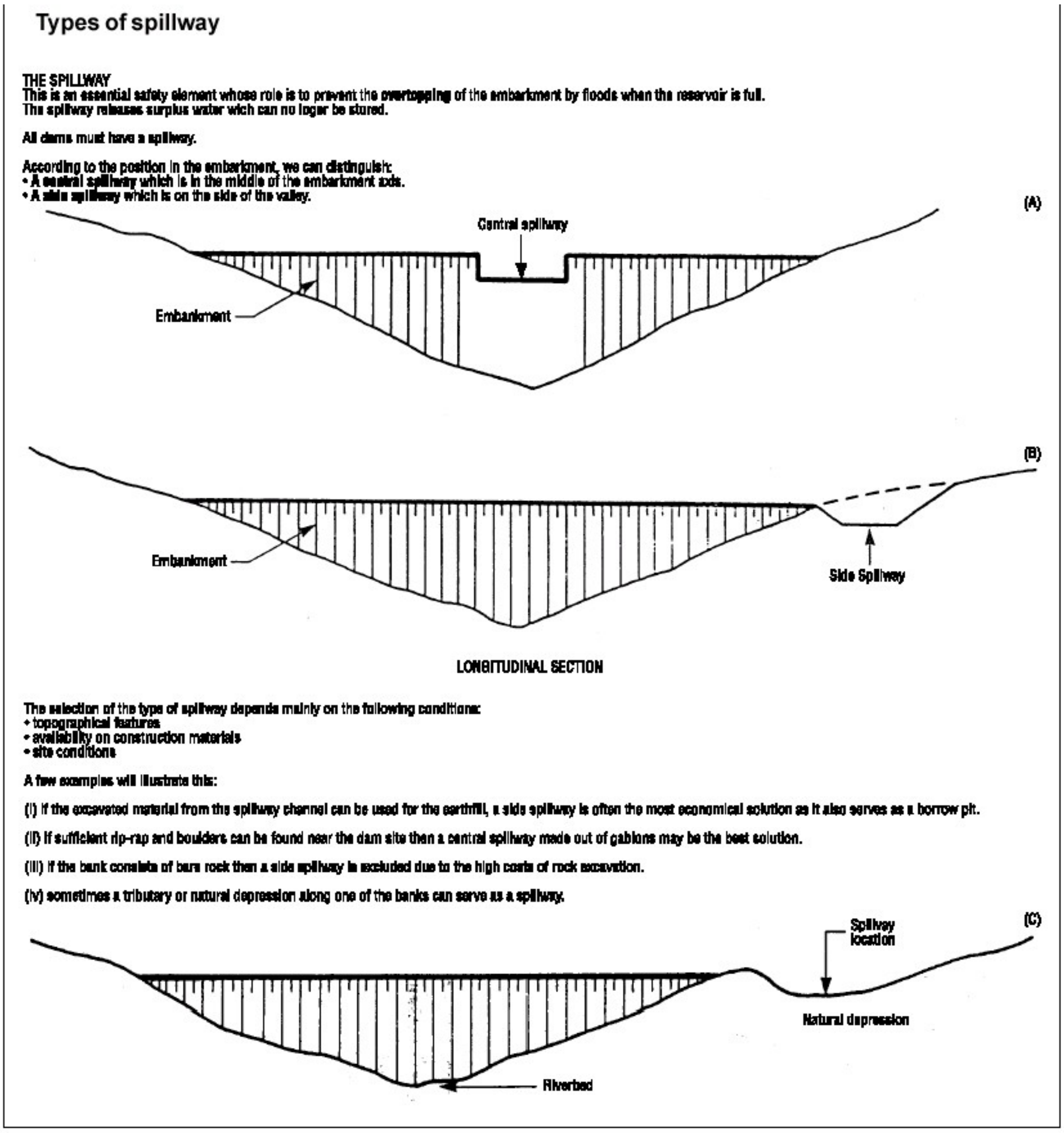
Classification of spillways

There are several types of spillway design. A general classification of spillways used in small hydraulic works is provided below. This classification focuses on the spillway's position in relation to the earthfill and to the valley's principal stream (see figure 3.1):

A – at the earthfill's centre, on the axis of the main stream,
B - lateral to the earthfill, out of the axis of the main stream,
C - external, out of the axis of the main streambed, discharging into a secondary side valley.

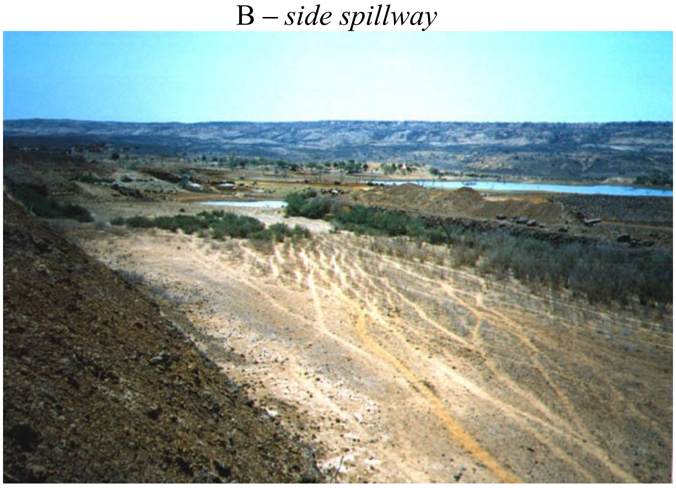
The first spillway typology (A – ***central spillway***) is characteristic of all kinds of gabion weirs (diversion weir, debris/check dams and water spreading dams). The spillway is a simple gabion weir with a stilling basin on its downstream side. It is generally inserted in the earthfill embankment.

The other two spillway typologies are characteristic of dams. In the first case (B – ***side spillway***) the spillway is positioned sideways the earthfill embankment, in the second (C – ***lateral spillway***), the spillway discharges the excess flow into a secondary lateral valley. The last solution is the most appropriate for small dams, because its cost is generally inferior to that of the other types. Another fundamental advantage of this spillway type (C) is the complete independence that it realises between earthfill and spillway, the main structures composing the hydraulic work, which can therefore be built at different moments in time. Otherwise, it would be necessary to build these two structures simultaneously, having to cope with all the problems of works co-ordination. Finally, this solution keeps water from flowing nearby the earthfill embankment, eventually causing problems in the area of contact between earthfill and gabion structures, especially if these have not been executed thoroughly.



A – *central spillway*

B – *side spillway*



B – *lateral spillway*

Fig. 3.1 – Types of spillway (A, B and C)

The spillway is generally composed (B and C) by a ***channel***, that carries excess water from the impoundment to the exit, and by a ***drop system***, for the water's restitution to the natural streambed. In the first spillway typology (A), the channel is absent and the excess flow is directly evacuated in the drop system.

The channel is generally excavated in the natural soil, and its characteristics (i.e. shape, slope and dimensions) have to be adequately predisposed to evacuate the design flow. The drop system can be composed of one or more weirs, according to the entity of energy dissipation and to the characteristics of the materials used in weirs building. Sometimes, a stilling basin is realised downstream the weirs in order to concentrate the water's energy dissipation in this zone.

3.2 – DISCHARGE CHANNEL OF THE SPILLWAY

Hydraulic design
Protection of channel sides and bottom

Various kinds of problems could be avoided if the spillway's discharge channel is accurately designed. The channel's principal characteristics are:

- transversal section and longitudinal slope both apt to evacuate the design flow,
- lining to prevent bank and bed erosion caused from water flowing,
- protection of the bank (side slope and gutters).

The dimensions of the transversal channel section and the longitudinal slope have to be calculated in accordance with the design flow in order to limit the water's speed. It is always preferable to opt for a large channel with a slight slope, rather than a narrow channel with a steep slope. Even if the first solution is somewhat more expensive to be realised, it normally proves cheaper later, because it requires a minimal upkeep with low related maintenance costs.

If the channel bed's soil materials cannot support the water flow, the channel will have to be ***lined*** with more resistant materials. In this case, the channel bank should be protected with a small ***gabions retaining wall***. ***Gutters*** must be built to prevent channel bank erosion caused by the runoff coming from upstream the hill in which the channel is built (see figure 3.2).

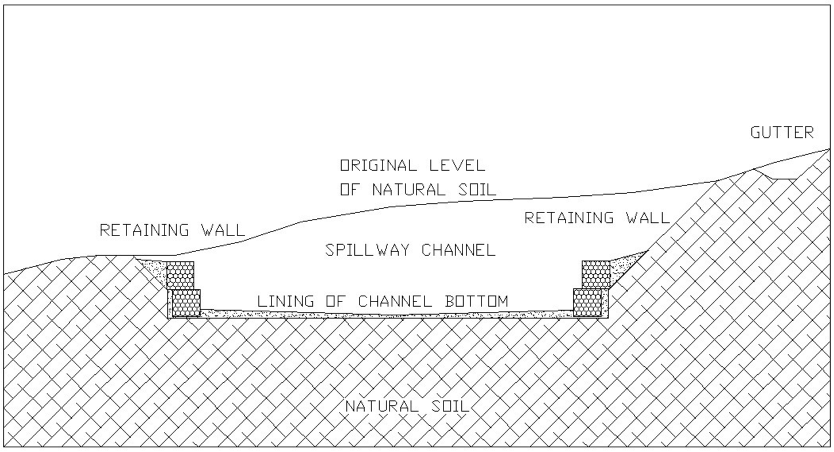


Fig. 3.2 - Cross section of spillway channel

3.2.1 – Hydraulic design

The spillway should be commensurate to the design flow. As mentioned above, the spillway is generally composed by a discharge channel and by a drop system: both these components have to be dimensioned according to the design flow. The dimensioning of a drop system will be explained in the following paragraph (3.3). Here the stream-flow computation for designing the discharge channel will be briefly explained (U.S. Department of the Interior Bureau of Reclamation. 1987).

In an open channel, in which, by hypothesis, flow streamlines are parallel and the speeds of all the points in a cross section are equal to the mean velocity v, water's energy consists of two components: kinetic and potential (Bedient P., Huber W. 1987). With reference to figure 3.3, the absolute head of an open channel's flow discharge is expressed by Bernoulli's equation:

$$H_a = z + d + v^2 / (2 \times g) \qquad \text{(absolute head)}$$

whereby:

z channel bottom level,
d water depth,
v mean velocity,
g gravity acceleration.

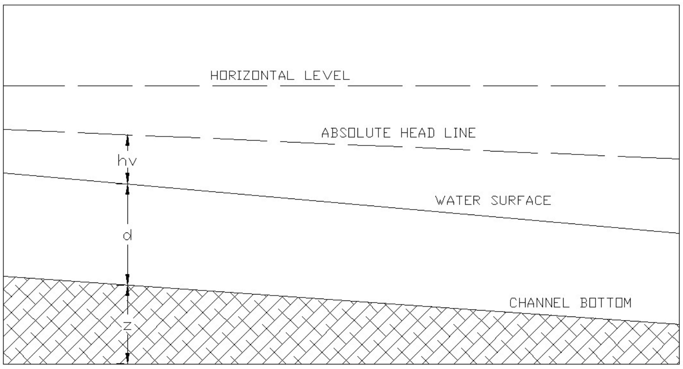


Fig. 3.3 - Flow in a channel

The energy at the channel's bottom, named ***specific energy***, is represented by the relation:

$$H_e = d + v^2 / (2 \times g) \qquad \text{(specific energy)}$$

the velocity v in a open channel can be expressed by:

$$v = Q / \Omega$$

whereby:

Q discharge (volume rate of flow),
Ω cross sectional area of flow,

then, the specific energy can also be expressed as

$$H_e = d + Q^2 / (2 \times g \times \Omega^2) \qquad \text{(specific energy)}$$

It is crucial to understand this formula correctly. The relation can be plotted with respect to the specific energy (H_e) and the water depth (d) axes (see figure 3.4), for different discharge (Q) values. The diagram shows that, for fixed discharge values Q_f and specific energy H_f, there are two possible depths d_1 and d_2, where d_1 is related to a ***sub-critical flow*** and d_2 to a ***super-critical flow***.

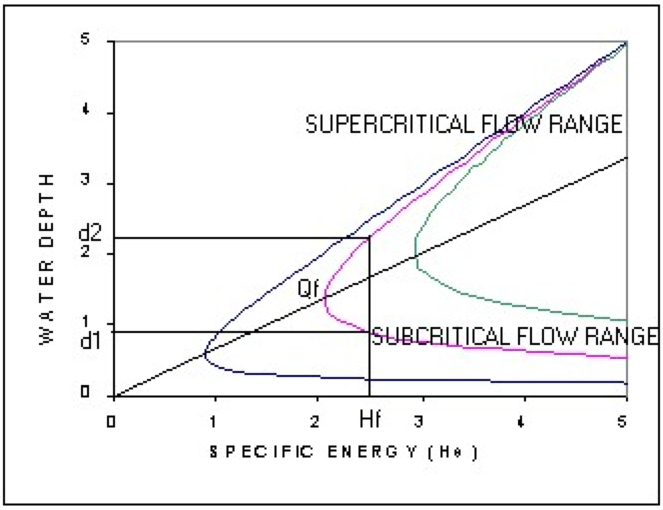


Fig. 3.4 – Flow's depth-energy relationship in the channel

$H_f = d_1 + v_1^2 / (2 \text{ x } g) = d_2 + v_2^2 / (2 \text{ x } g)$

with

$d_1 < d_2$ and $v_1^2 > v_2^2$

It can be seen that in the supercritical flow range, the water velocity is always higher than in the subcritical flow range. There is also a minimal specific energy (H_m), to which corresponds a unique value of water depth (d_m). When this condition occurs, the flow is named critical, as well as all the other characteristics (i.e. depth, velocity and slope).

The key parameter used to express the discharge flow condition is the ***Froude number***:

$F_r = v / (g \text{ x } y_m)^{0.5}$

whereby y_m is the average flow depth. If

$F_r > 1$, then the discharge flow is in supercritical conditions,
$F_r < 1$, then the discharge flow is in sub-critical conditions.

Establishing whether a flow falls in the supercritical or the sub-critical range is very important. Firstly because, in small hydraulic works, all the structures should be designed, if possible, so as to keep the flow in the sub-critical range. In fact, problems caused by water's erosion are reduced at the sub-critical range's lower flow speed.

Figure 3.5 shows what happens when a discharge flows in the spillway channel of an impoundment. At the channel's entrance, there is a ***transitional zone*** to which corresponds a loss of specific energy due to entrance frictions. Simultaneously, the water starts flowing in the channel and the energy passes from potential to kinetic. In the impoundment, the kinetic component of specific energy is generally negligible and the specific energy line corresponds to the water level. After the transitional zone, the discharge flow is in a uniform condition (***uniform zone***), with the water level parallel to the channel bottom. In this zone, it is always preferable to keep the flow at a sub-critical level, to avoid scour problems in the channel. Otherwise, it will be necessary to line the channel. Close to the final drop, at the end of the channel, the flow's speed rises and the water level decreases until it reaches the critical condition on the drop (***transitional zone***). The following paragraph illustrates what happens beyond the drop.

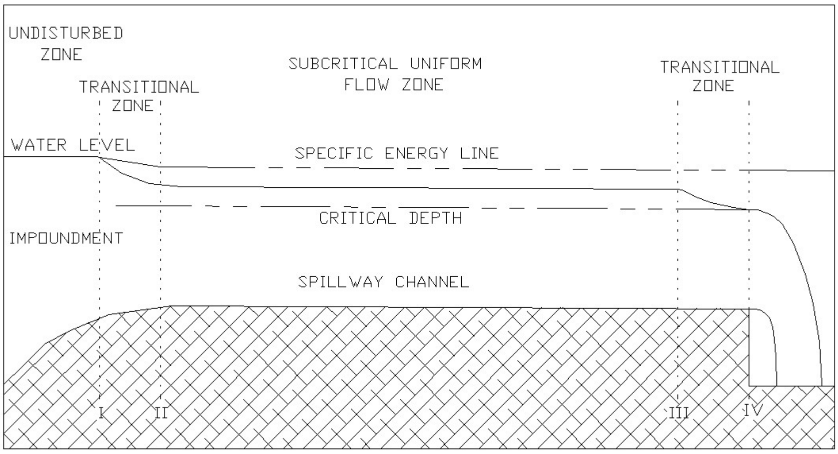


Fig. 3.5 - Schematisation of a spillway channel

The design of a spillway channel should aim at minimising specific energy losses and at limiting the water's speed to prevent scour problems. Minimising specific energy losses will allow us to design a higher channel bottom, thus reducing the amount of earthworks. Reducing water erosion will allow us to shield the channel bed with a simple and superficial lining (e.g. with a thin layer of rubble and stones). If the natural soil is of a resistant quality, channel lining will not be necessary. In order to minimise energy losses at the channel entrance, a ***progressively straitening funnel-shaped entrance*** should be substituted to the natural entrance. If the new funnel-shaped entrance is properly realised, specific energy losses will amount to a few centimetres only.

Where a uniform flow is achieved, after the transitional zone, the usual flow-depth relationship can be used for calculating the water depth corresponding to the design flow. For this computation, the channel bottom's slope and its roughness in relation to the materials lining the channel should be fixed in advance. One of the relationships used most frequently for flow computations in open channels is ***Manning's formula*** (Bedient P., Huber W. 1987, FAO. 1996 , Maccaferri 1990a):

$$v = 1 / n \times R^{2/3} \times i^{1/2} \qquad (\text{with } k = 1 / n)$$

which derives from ***Chezy's general formulation***:

$$v = X \times (R \times i)^{1/2}$$

whereby:

n	Manning's coefficient,
k	Gaukler-Strikler's coefficient,
R	hydraulic radius,
i	spillway channel longitudinal slope,
X	Chezy's coefficient.

The best way to use Manning's relationship is to make repeated attempts with different water level values in order to compute the flow's velocity and, consequently, the flow's discharge, using the relation

$$Q = v \times \Omega$$

until, in the spillway channel, the water level that allows the passage of the design flow is eventually found. The values of Manning's (n) and Gaukler-Strickler's (k) coefficients are tabulated relatively to the channel bottom and bank materials (see figure 3.6).

Roughness coefficient of Manning

	MIN.	NORM.	MAX.
Minor streams (top width at flood stage < 30 m)			
a) *Streams on plain*			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools, some weeds and stones	0.033	0.043	0.050
4. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
5. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
6. Very weedy reaches, deep pools floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b) *Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages*			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Flood plains			
a) *Pasture, no brush*			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b) Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c) Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in summer	0.040	0.060	0.080
3. Medium to dense brush, in summer	0.045	0.085	0.160
d) Trees			
1. Dense willows, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major streams (top width at flood stage > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance			
1. Regular section with no boulders or brush	0.025		0.060
2. Irregular and rough section	0.035		0.100

Fig. 3.6 – Coefficient values for flow relations in open channels

When the value of the water level required to evacuate the design flow in the spillway channel has been established, a verification of water levels in the channel entrance is necessary. With respect to figure 3.5 , the sum of absolute head and energy losses in section II must be inferior to the absolute head in section I, otherwise the discharge rate in the channel will be lower than the design flow.

$$H_I = z_I + h_I$$
$$H_{II} = z_{II} + h_{II} + v_{II}^2 / (2 \times g)$$
$$H_I > H_{II} + \Delta H$$

ΔH is negligible if the channel entrance is well realised. If this relation is not verified, then the spillway channel's characteristics should be modified (i.e. augmentation of width or slope, modification of channel lining in order to diminish its roughness).

If the compatibility of absolute heads expressed in the above relation is satisfied, then the ***channel bed's resistance*** to scour must be verified. The maximum flow speed values that do not provoke erosion are tabulated in figure 3.7 for several materials.

Maximum permissible velocities for different materials

Material	Clear water V (m/sec)	Water transporting colloidal silts V (m/sec)
Fine sand, colloidal	0,45	0,76
Sandy loam, noncolloidal	0,53	0,76
Silt loam, noncolloidal	0,60	0,91
Alluvial silts, noncolloidal	0,60	1,06
Ordinary firm loam	0,76	1,06
Volcanic ash	0,76	1,06
Stiff clay, very colloidal	1,14	1,52
Alluvial silts, colloidal	1,14	1,52
Shales and hardpans	1,82	1,82
Fine gravel	0,76	1,52
Graded loam to cobbles when noncolloidal	1,14	1,52
Graded silts to cobbles when colloidal	1,22	1,67
Coarse gravel, noncolloidal	1,22	1,82
Cobbles and shingles	1,52	1,67

(For sinuous channels, the velocities should be lowered. Percentage of reductions suggested by Lane vary from 5% for moderately sinuous to 22%, for very sinuous channels).

Fig. 3.7 : Maximum water speed tolerated by different materials

These values have been extrapolated from different theories (e.g. Shields' diagram, which imposes a fixed value for the critical shear stress), with the support of experimental observations (U.S. Department of the Interior Bureau of Reclamation. 1987 Maccaferri 1990a). If the calculated water speed in the spillway channel is higher than the maximum tolerable speed for the bottom material, taken from figure 3.7, then the spillway channel's design should be modified. Two are the possibilities:

- augmentation of channel width and/or diminution of channel slope in order to lower the water speed,
- channel lining with proper materials which resist to the calculated water velocity.

After having modified the spillway channel's characteristics in one of the two possible ways that have just been mentioned, a new test to ascertain that the water speed is inferior to the maximum speed tolerable by the channel bed's material is necessary. Then, the verification expression, with new absolute heads' values, will have to be reformulated because the channel's hydraulic characteristics have now changed.

When the spillway channel is very short, its realisation in ***reverse slope*** is generally preferable, especially if the natural soil materials are not particularly resistant to water flow. With a reverse slope, the water speed in the channel is lower than in the vicinity of the final weir and the risks of channel erosion are reduced. Moreover, the reverse slope also prevents water stagnation in the discharge channel. Especially in arid and semi-arid regions, water stagnation facilitates the growth of vegetation, introducing new maintenance requirements.

3.2.2 – Protection of channel sides and bottom

Shields' diagram allows us to calculate the minimal size at which particles are not transported by water flow in the channel. If the natural soil material contains a percentage of particles of a size smaller than that calculated through Shields' diagram, the water flow can give rise to an important scour phenomenon in the channel. In this case, the channel bed will have to be lined with a more resistant material, e.g. containing a higher percentage of gravel, rubble and stone, and a small percentage of sand and clay. This material must be properly graded in order to obtain a high percentage of particles (between 80 and 90 %) with a diameter larger than the one computed with Shields' diagram. Some ***lines of gabions*** transversal to the channel can be inserted to prevent bed scour. The gabions' level should be positioned a few centimetres above the channel bed's lining, as shown in fig. 3.8.

In humid zones a ***turfing protection*** can be used for lining the spillway channel.

Fig. 3.8 - Linear protection of gabions on the channel bottom

If the channel crosses layers of material particularly vulnerable to water erosion, the spillway channel sides should be protected with ***retaining gabion walls*** to prevent banks erosion. The retaining walls at the sides of the spillway channel will also serve the purpose of stabilising the channel bank slopes made of incoherent materials.

The retaining walls cross section must be calculated according to earthfill and water stresses. The procedures for verifying the stability of the retaining walls are similar to the ones used to verify weir stability, which will be explained in the following paragraph (3.3). Figure 3.9 shows two possible ways to realise the cross section of retaining walls. Option A is to be preferred if banks are made of rather resistant materials and the earthfill is properly compacted. In all other cases, option B will be more convenient.

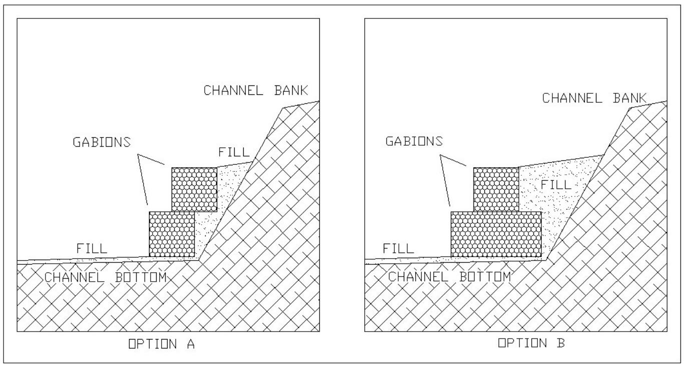


Fig. 3.9 - Retaining walls (options A and B)

Gabions retaining walls establish a ***preferential flow path***, especially in the areas close to the bottom and the sides of gabions. In these areas, the internal conformation of gabions, with rubbles and channels, can cause the acceleration of the water flow. The erosive potential of water is increased, and the finest particles of material in contact with the gabion can be removed. This erosion phenomenon, peculiar to the areas in the vicinity of gabions, causes settlements of the wall, eventually leading to its failure. The most effective techniques to prevent erosion problems will be mentioned here very briefly. A detailed illustration of these techniques, and of the procedures to follow for their realisation, can be found in section 4.

The problem of erosion in the ***contact zones*** between gabions and natural soils or artificial earthfill embankment is common to all the hydraulic structures that include gabions (i.e. retaining walls, weirs, counter-weirs). To limit this problem, it is possible to resort to several solutions, two of which are particularly effective:

- interposition of geotextile between gabions and natural soil or artificial earthfill,
- building of semi-permeable or impervious cut-offs.

The first solution is the most suitable one. In fact, a ***geotextile layer*** should always be placed at the interface between gabions and natural soil or artificial earthfill, when the gabions structure can be interested by water passage. If the passage of water throughout the gabion structure is critical, then it is preferable to build ***semi-permeable*** or ***impervious cut-offs***. These cut-offs should be built transversal to the flow direction to reduce water's erosive power. Semi-permeable cut-offs are realised interposing a geotextile layer between two layers of gabions. Impervious cut-offs, instead, take the form of a concrete wall, or walled gabions (gabions realised with a particular technique).

Above the banks of the spillway channel, a ***gutter*** should be installed to drain and to evacuate runoff water coming from upstream the channel, which, left unchecked, could lead to bank erosion (see figure 3.2).

3.3 – WEIRS IN THE SPILLWAY

- ***Shape of the weir’s downstream side***
- ***Type of weir***
- ***Hydraulic design***

As shown in the previous paragraph, before reaching the drop, the discharge flow generally falls in the subcritical range, right on the drop the flow is critical, and beyond the drop it becomes supercritical (see figure 3.10). Here, water flowing in the supercritical range erodes the streambed, downstream the weir, due to the progressive energy dissipation that follows the flow discharge.

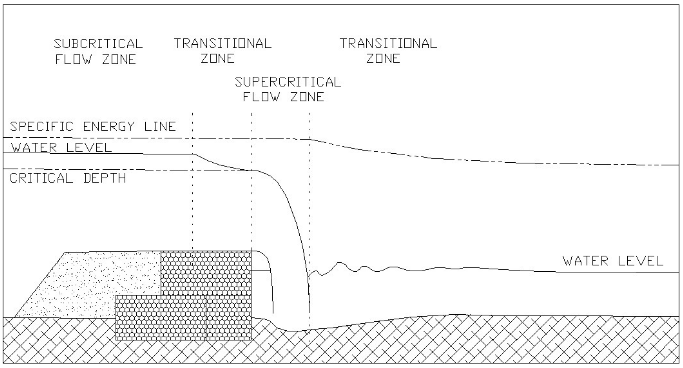


Fig. 3.10 – Water flowing on a weir

In this paragraph, weirs are firstly classified according to shape (U.S. Department of the Interior Bureau of Reclamation. 1987, Maccaferri 1990a). Then, weirs will be classified according to their hydraulic function with reference to the problem of energy dissipation. Hydraulic computations will be presented in the third part of this paragraph with regard to vertical weirs. In fact, a vertical downstream side is the most common in small hydraulic works. Finally, the stability computation for gabions structures will be explained in the fourth part of the paragraph.

3.3.1 - Shape of the weir’s downstream side

Weirs can be classified, according to the shape of their downstream side, into three classes (as shown in fig 3.11):

- *vertical*
- *stepped*
- *battered*

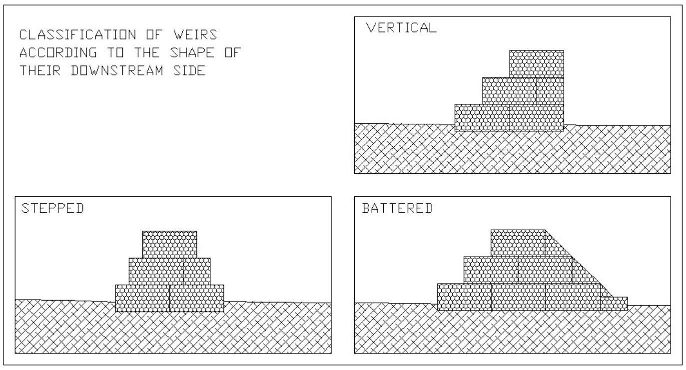


Fig. 3.11 - Weir shapes

The shape of the downstream side of a weir must be chosen according to several factors, such as:

- drop height,
- hydraulic charge,
- characteristics of the materials employed in gabions building,
- characteristics of local natural soils,
- weir class according to the first classification mentioned above (in respect to the presence of a stilling basin).

There are no general rules for the choice of a particular shape. However if the drop's height does not exceed 3÷5 m, the vertical shape is always the most appropriate.

If the drop height is higher than 3÷5 m, the weir must be designed with a stepped shape, which, however, can be used only if the specific flow does not exceed 3 $m^3/s/m$.. Otherwise the turbulence and the shock provoked by water dropping on the step can bring about severe breakage in the gabions. In Cemagref's interesting synthesis of experimental observations on stepped gabions weirs, it is argued that if the drop is higher than 3÷5 m, and the specific flow does not exceed 1 $m^3/s/m$, the battered shape would also be suitable (Peyras , Royet , Degoutte. 1991).

Step- and batter- shaped weirs are not well fitted to natural streams with a significant solid transportation, especially when gravel and rubber are transported by the flow. In fact, the continuous dropping and sliding of particles on and through gabions can provoke net tears.

In section 4, a number of techniques that can be adopted to face gabions failure will be illustrated for each kind of weir shape.

The weir's upstream side should always be stepped in order to facilitate the bonding between gabions and earthfill, which functions so as to make the weir impervious. Moreover, earthfill's weight on the weir steps adds stability to the structure, contributing to prevent sliding and overturning events (see paragraph 3.3.4).

3.3.2 - Type of weir

Sometimes, natural flow conditions downstream the structure, can provoke a concentration of energy dissipation in a circumscribed zone. Otherwise, a structure on purpose (***stilling basin***) will have to be realised downstream the weir.

With reference to the problem of energy dissipation downstream the structure, weirs can be classified in four categories (U.S. Department of the Interior Bureau of Reclamation. 1987, Maccaferri 1990a) (see figure 3.12).

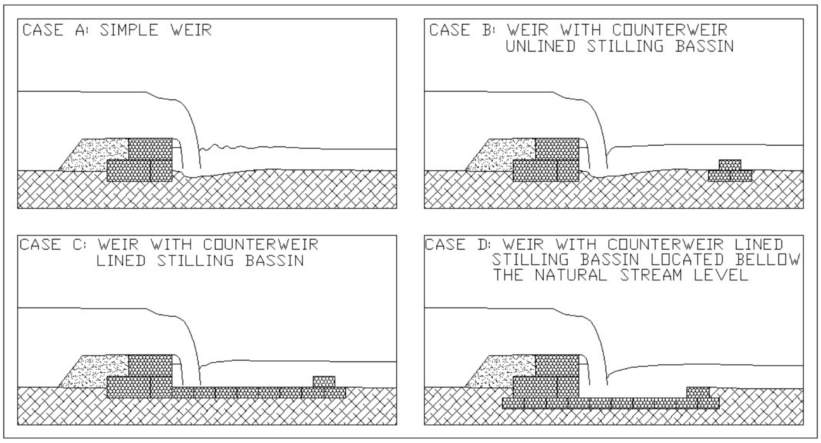


Fig. 3.12 - Type of weirs (case A, B, C and D)

- *simple (A),*
- *with counterweir, unlined stilling basin (B),*
- *with counterweir, lined stilling basin (C),*
- *with counterweir, stilling basin located below the natural river bed (D).*

The methods used to ascertain that the energy dissipation is concentrated immediately downstream the weir will be illustrated in paragraph 3.3.3.

3.3.3 - Hydraulic design

- *Simple weir*
- *Weir with counterweir and unlined stilling basin*
- *Weir with counterweir and lined stilling basin*
- *Weir with counterweir and lined stilling basin located below the natural river bed*
- *Stepped weirs*
- *Battered weirs*
- *Verification against piping failure*

The procedures explained below are primarily related to the hydraulic design of weirs with a vertical downstream side. Vertical weirs are the simplest to design and to build, accounting for their widespread use as small hydraulic works, especially in developing countries. Hydraulic dimensioning procedures for stepped and battered weirs will be only briefly mentioned. It should be noted, however, that some weirs with a stepped downstream side, can be dealt with in the same way as vertical weirs, with respect to hydraulic computations, if the downstream side slope is so important that water jumping from the weir crest does not flow on the steps but falls straight upon the weir toe, downstream.

At the initial stages of weir design, the only known characteristic is its height. The weir's height depends upon the difference between design slope and original slope. In order to limit erosion problems, if the weir is expected to be taller than 2÷4 meters it could result useful to build more than one weir, depending on the natural soils characteristics and on the quality of construction materials (see figure 3.13).

Fig. 3.13 - Channel longitudinal section with weirs

The flow-depth relationship in Chezy's formula

$Q = b \times h \times \mu \times (2 \times g \times h)^{0.5}$

allows the calculation of the water's depth atop the weir (h), provided that design flow (Q), discharge coefficient (μ) and weir's width (b) are known.

Discharge coefficient values are tabulated according to water charge (h) and weir crest length (see figures 3.14 and 3.15) (Cremonese. 1996).

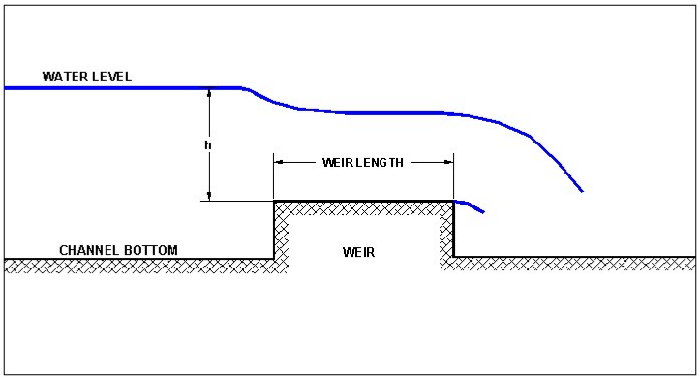


Fig. 3.14 - Weir schematisation

Discharge coefficients

Head (m)	Weir length (m)									
	0,15	**0,30**	**0,45**	**0,60**	**0,75**	**0,90**	**1,00**	**1,20**	**3,00**	**4,50**
0,06	0,347	0,335	0,327	0,317	0,309	0,304	0,297	0,292	0,310	0,334
0,12	0,364	0,340	0,329	0,325	0,324	0,322	0,317	0,312	0,319	0,337
0,18	0,384	0,343	0,329	0,325	0,324	0,334	0,335	0,337	0,337	0,337
0,24	0,411	0,355	0,334	0,324	0,324	0,333	0,334	0,334	0,335	0,329
0,30	0,414	0,372	0,343	0,332	0,329	0,330	0,333	0,334	0,334	0,328
0,36	0,414	0,384	0,357	0,337	0,330	0,329	0,333	0,332	0,335	0,329
0,42	0,414	0,399	0,364	0,345	0,334	0,329	0,330	0,330	0,333	0,329
0,48	0,414	0,409	0,373	0,360	0,343	0,334	0,332	0,330	0,329	0,328
0,54	0,414	0,412	0,383	0,359	0,342	0,334	0,332	0,330	0,329	0,328
0,60	0,414	0,411	0,383	0,355	0,344	0,340	0,334	0,330	0,329	0,328
0,75	0,414	0,414	0,409	0,383	0,360	0,350	0,340	0,333	0,329	0,328
0,90	0,414	0,414	0,414	0,399	0,380	0,364	0,341	0,332	0,329	0,328
1,05	0,414	0,414	0,414	0,414	0,398	0,370	0,344	0,334	0,329	0,328
1,20	0,414	0,414	0,414	0,414	0,414	0,383	0,348	0,337	0,329	0,328

For submerged spillways (downstream water level higher than the crest level) apply corrective coefficient given in Table 5.4

Fig. 3.15 - Discharge coefficient values

Discharge corrective factor for submerged weirs

Fawer formula for submerged weirs: $Q = mCB\sqrt{2gH3/2}$

a/Ho	0,00	0,10	0,20	0,30	0,40	0,50	0,60	0,70	0,80	0,90	1,00
C	0,10	0,99	0,96	0,92	0,88	0,81	0,71	0,52	0,40	0,32	0,00
a/Q2/3	0,00	0,10	0,20	0,30	0,40	0,50	0,60	0,80	1,00	1,20	1,40
C	1,00	1,00	0,99	0,96	0,92	0,88	0,81	0,71	0,52	0,40	0,32

a: Downstream hydraulic head above the crest
Ho: Upstream hydraulic head above the crest

The ***discharge coefficient*** range is wide: its values generally vary between 0.3 and 0.4. Choosing the right value for the discharge coefficient is not easy, as it also depends on the crest's roughness conditions, which may vary during runoff (e.g. if shrubs carried by the flow get trapped in the gabions net). It should be noticed that the values of the discharge coefficient are not available for submerged weirs (e.g. downstream water level higher than the weir's crest).

The engineer designing a weir, should chose a limit value for the discharge coefficient in order to maximise the water depth with a fixed flow design. For example, for a gabions weir completely filled with sediments upstream, and with a water depth between 1 and 2 metres, a prudential discharge coefficient value would be 0.35.

The maximum water depth should be kept within a limited range of values, with a peak value of 2÷3 metres, modifying weir's width accordingly, if necessary. The problems that could arise in a weir when the water charge exceeds 2 metres are discussed in chapter VII. This chapter will introduce building solutions that allow the weir to support greater water charges (i.e. reinforced concrete lining of the stilling basin).

After having fixed the weir's main dimensions (e.g. height, width, water charge) it is necessary to verify what happens downstream the structure. As we have seen, the engineer must ensure that the energy dissipation is concentrated immediately downstream the structure. This condition is satisfied when a subcritical flow, with particular characteristics, takes place downstream the structure.

It is also important to make sure that, between this sub-critical flow and the flow coming out from the weir (generally supercritical) the conditions for the installation of a ***hydraulic jump*** are fulfilled. Several methods can be adopted for proceeding to this verification. The choice of the most appropriate method depends on the weir type, selected according to the classification suggested in paragraph 3.3.2. These methods and the procedure for dimensioning a stilling basin are explained below (U.S. Department of the Interior Bureau of Reclamation. 1987, Maccaferri 1990a).

Simple weir

For a very small weir with limited specific flow and energy dissipation, the gabions structure can be realised omitting the stilling basin. This solution is likely to be chosen especially if the streambed material is resistant. Otherwise dropping water could dig a hole downstream the weir. In this case, it will be necessary to calculate hole depth and the distance between weir structure and hole.

With reference to figure 3.16, considering that the flow, on the weir, falls in the critical range, the distance X can be computed through the following rough relation:

$$X \cong (2 \times (z_g - f_g) \times (z_g - f_3))^{0.5}$$

Scour depth can be computed with Schoklitsch's relation expressed, always with respect to figure 3.16, by:

$$z_3 - f_b = 4.75 \times (z_0 - z_3)^{0.2} \times q^{0.57} / d_t^{0.32}$$

where z_3, f_b, z_0 are expressed in meters, q, expressed in $m^3/s/m$, represents the specific flow, and d_t is the sieve diameter through which passes the 90% of streambed material. For safety reasons, the weir's foundation level should be lower than the hole's minimum level.

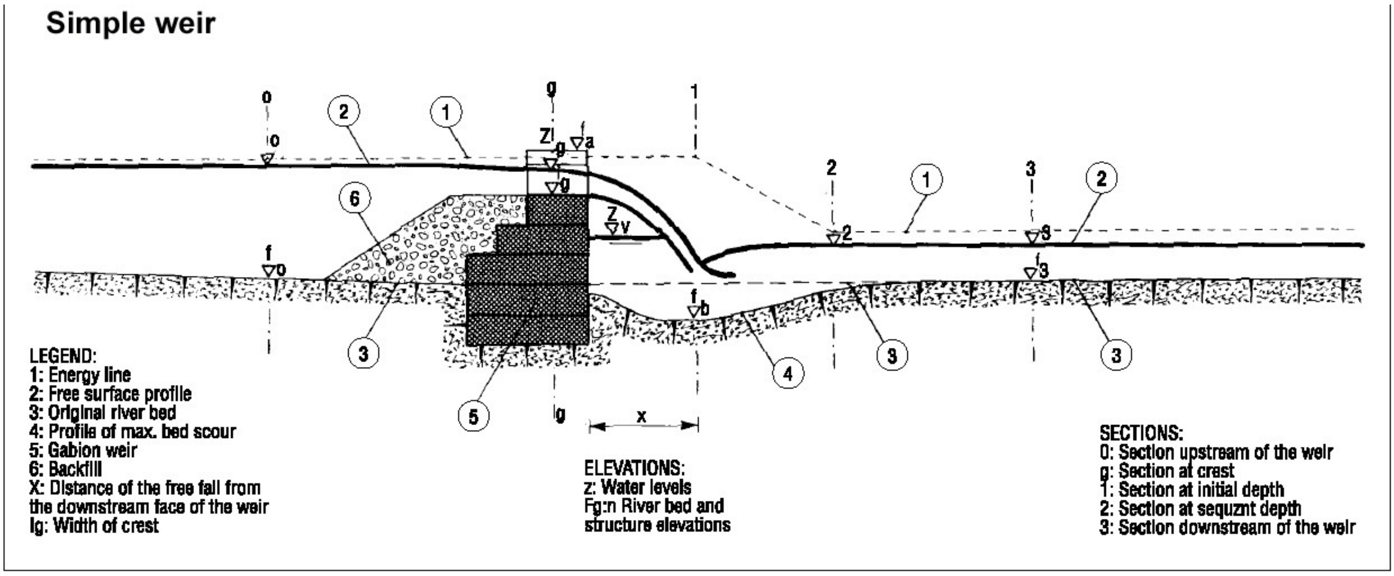


Fig. 3.16 - Simple weir

Weir with counterweir and unlined stilling basin

A ***counterweir*** can be built in order to reduce the erosion phenomenon caused by energy dissipation downstream the weir. This will cause the water level downstream the weir to increase, and the scour depth will be reduced.

With regard to the weir, the counterweir has to be placed at a distance and at a level which, in subcritical conditions, allow the formation of a flow discharge. The counterweir's height will be calculated by means of the usual depth-flow relationship (see figure 3.17):

$$Q = lc \times (z_2 - f_c) \times \mu \times (2 \times g \times (z_2 - f_c))^{0.5}$$

whereby:

lc weir width,
μ discharge coefficient,
g gravity acceleration.

z_2 should be preventively assigned a value which limits the scour's depth, then the above relation will be used for computing f_c.

For computing the distance between weir and counterweir, the evaluation of hydraulic jump length is necessary throughout the relation:

$$L_{12} \cong 6.9 \times (z_2 - z_1)$$

then the total length of stilling basin will be:

$$L_t = L_{12} + X$$

with X's value so as calculated above, for simple weir.

When a counterweir is being realised, the energy dissipation downstream the structure should always be quantified and, if necessary, reduced in order to prevent the scour phenomenon in the streambed. To quantify energy dissipation, the computation of the hydraulic flow conditions downstream the counterweir is necessary. The flow equation in Manning's formulation can be used to compute water depth and velocity within the stream reach:

$$v = (1 / n) \times R^{2/3} \times i^{1/2}$$

and

$$Q = \Omega \times v$$

whereby:

n Manning's coeffcient,
R hydraulic radius,
i streambed slope,
Ω cross sectional area,
Q discharge flow.

To avail oneself of these relations, different water depth values should be tried until the correct discharge flow value is identified.

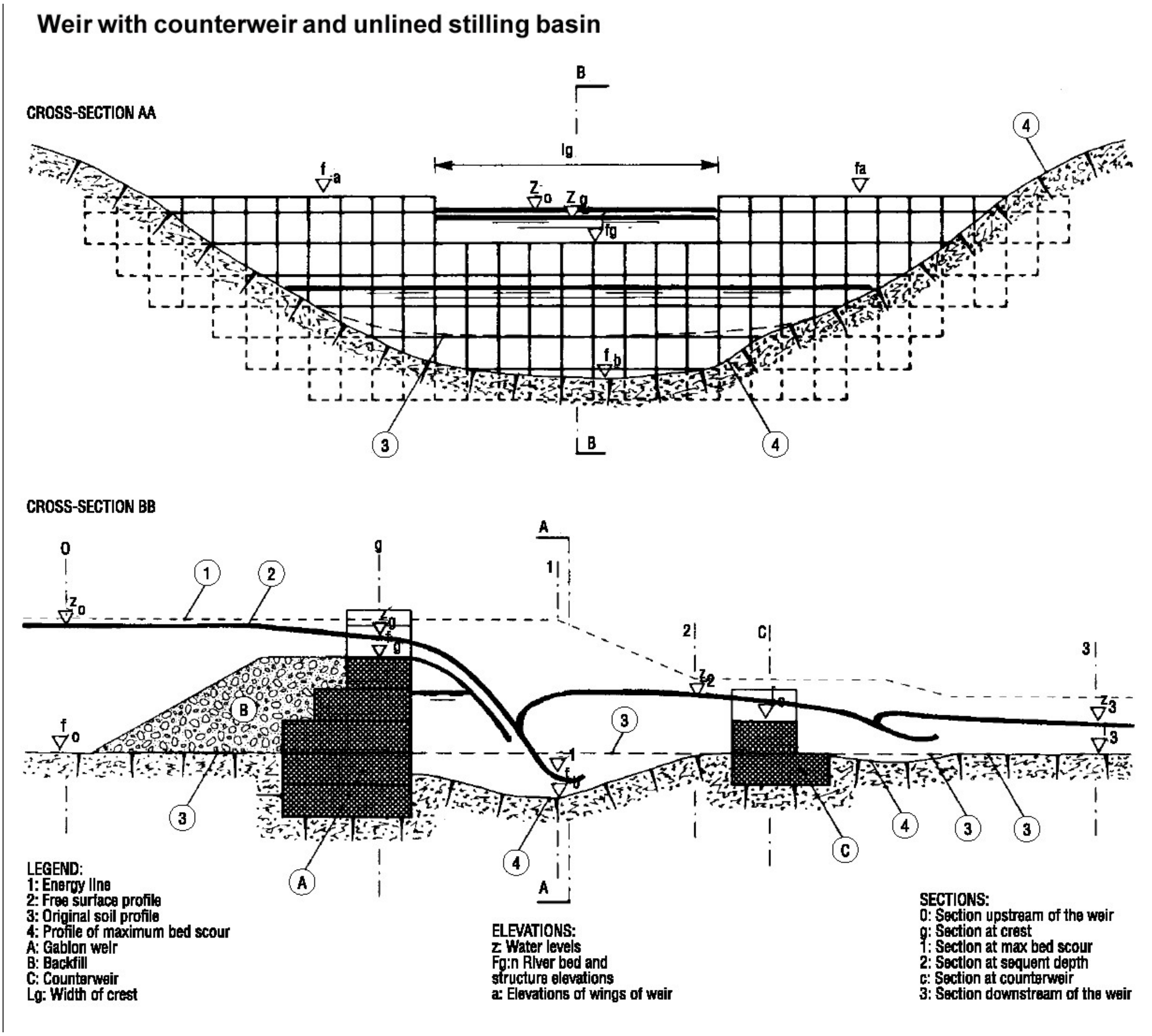


Fig. 3.17 - Weir with counterweir and unlined stilling basin

Weir with counterweir and lined stilling basin

If the streambed material is not very good (e.g. small grain sizes), the stilling basin will have to be lined in order to limit the weir's foundation level. For lining the basin's bottom a layer of gabions could be used, as shown in figure 3.18.

All the gabions structure's dimensions and water levels can be computed using flow-water's depth relations with the proper simplifying hypotheses. With reference to figure 3.18, the water depth of the supercritical flow is given by:

$$(z_1 - f_b) \cong \frac{Q}{l_b \times (2 \times g \times (z_0 - f_b))^{0.5}}$$

For energy dissipation to take place in the stilling basin, it should be seen that the hydraulic jump takes place. The water's depth of the subcritical flow is given by:

$$(z_2 - f_b) = - \frac{(z_1 - f_b)}{2} + \sqrt{\frac{2 \times Q^2}{g \times l_b^2 \times (z_1 - f_b)} + \frac{(z_1 - f_b)^2}{4}}$$

for obtaining this water depth, it will be expedient to realise a counterweir, the height of which can be computed through the usual depth-flow relationship:

$$Q = lc \times (z_2 - f_c) \times \mu \times (2 \times g \times (z_2 - f_c))^{0.5}$$

To complete our knowledge of water levels, the water depth in non-aerated zones can be obtained from the following relation:

$$(z_v - f_b) = (f_g - f_b) \times (Q^2 / (g \times l_b^2 \times (f_g - f_b)^3))^{0.22}$$

Before we can determine the stilling basin's length, we will have to calculate the distance from the weir at which the supercritical flow is installed and the length of the hydraulic jump. The former can be calculated as follows:

$$Lg_1 = \frac{(z_g + f_g - 2 \times f_b) \times (z_g - f_g)^{0.5}}{(z_g + f_g - 2 \times z_v)^{0.5}}$$

and the hydraulic jump length is given by:

$$L_{12} = 6.9 \times (z_2 - z_1)$$

At this point, we must verify that the stilling basin's flow behaviour is independent from the flow behaviour of the downstream reach. This will be confirmed if the total energy downstream is lower than on the counterweir.

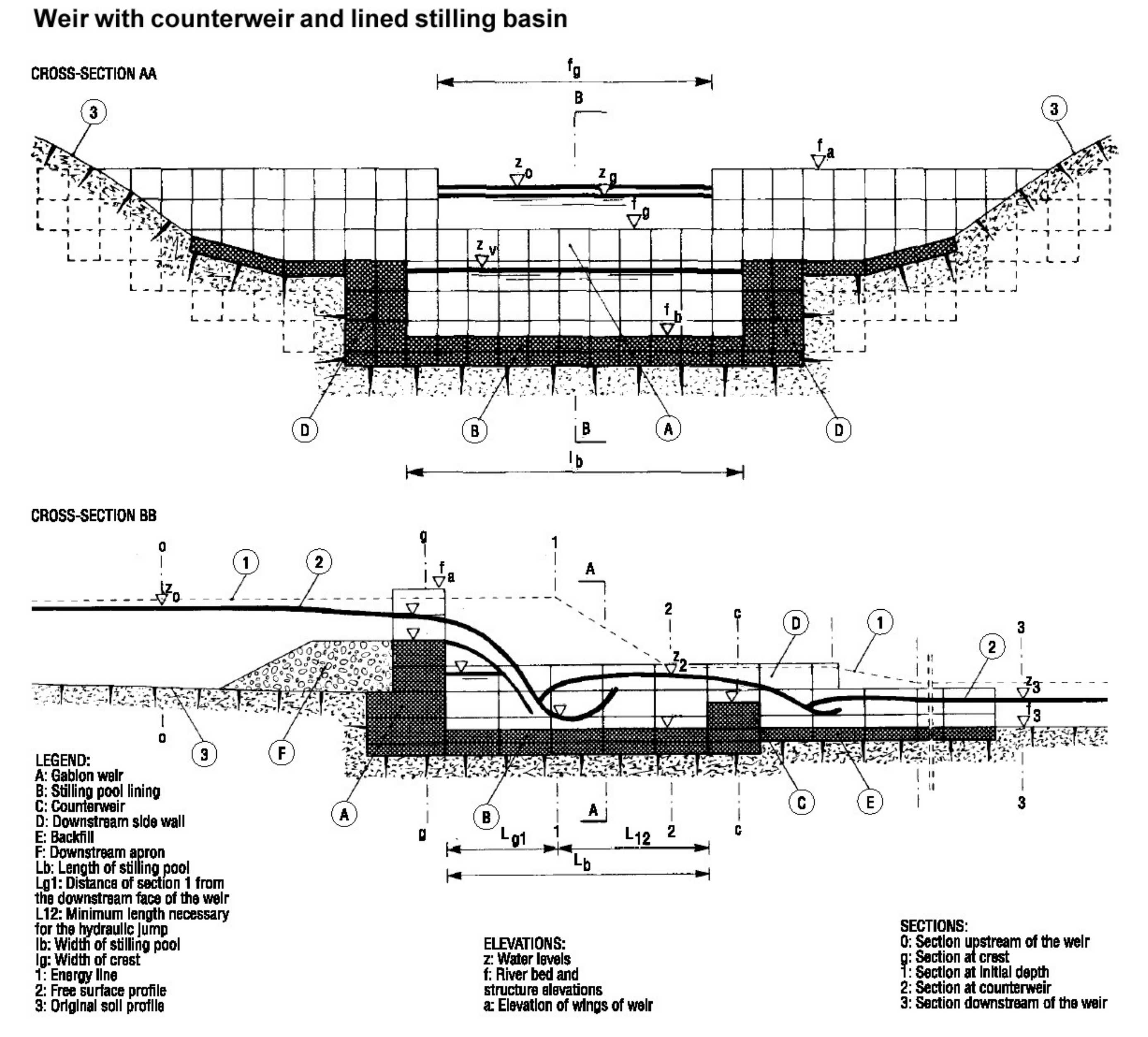


Fig. 3.18 - Weir with counterweir and lined stilling basin

When the upstream side of the weir is completely filled with sediment, existing sample relations allow us to determine all the characteristics of the stilling basin with reference to the ***drop number*** (D). These relations have been obtained by means of experimental observations. The drop number is expressed by:

$$D = \frac{q^2}{g \times (f_g - f_b)^3}$$

Once the drop number is known, all the characteristics of a weir with stilling basin can be obtained by applying the following relations:

$Lg_1 / (f_g - f_b) = 4.30 \times D^{0.27}$

$(z_v - f_b) / (f_g - f_b) = 1.00 \times D^{0.22}$

$(z_1 - f_b) / (f_g - f_b) = 0.54 \times D^{0.425}$

$(z_2 - f_b) / (f_g - f_b) = 1.66 \times D^{0.27}$

$L_{12} = 6.9 \times (z_2 - z_1)$

Sometimes, in order to reduce the quantity of gabions required for lining the stilling basin, it will be useful to place gabions lining only in the zone close to weir toe. The remaining portion of stilling basin will be lined with large stones (see figure 1.2). In this case, gabions lining has to extend itself up to a distance from the weir toe greater than Lg_1, to protect this portion of the stilling basing, which is threatened by water falling from the weir crest.

Weir with counterweir and lined stilling basin located below the natural river bed

In this case, the basin's functioning is influenced by the subcritical flow downstream. To obtain all the characteristics of the stilling basin, the composite system shown below must be solved.

$$(z_0 - f_b) + \frac{Q^2}{2 \times g \times \Omega_0^2} = (z_1 - f_b) + \frac{Q^2}{2 \times g \times (z_1 - f_b)^2 \times l_b^2}$$

$$(z_2 - f_b) = - \frac{(z_1 - f_b)}{2} + \sqrt{\frac{2 \times Q^2}{g \times l_b^2 \times (z_1 - f_b)} + \frac{(z_1 - f_b)^2}{4}}$$

$$(z_3 - f_b) + \frac{Q^2}{2 \times g \times \Omega_3^2} = (z_2 - f_b) + \frac{Q^2}{2 \times g \times (z_2 - f_b)^2 \times l_b^2}$$

Some of the flow and weir characteristics needed to solve the system are already known. The values of z_1, z_2 and f_b are the only unknown terms. It will be useful to preventively fix a value for f_b, in order to compute the value of z_1 in the first equation and the value of z_2 in the second one. If, at this point, the third equation is not satisfied, it will be necessary to restart the calculations with another value for f_b.

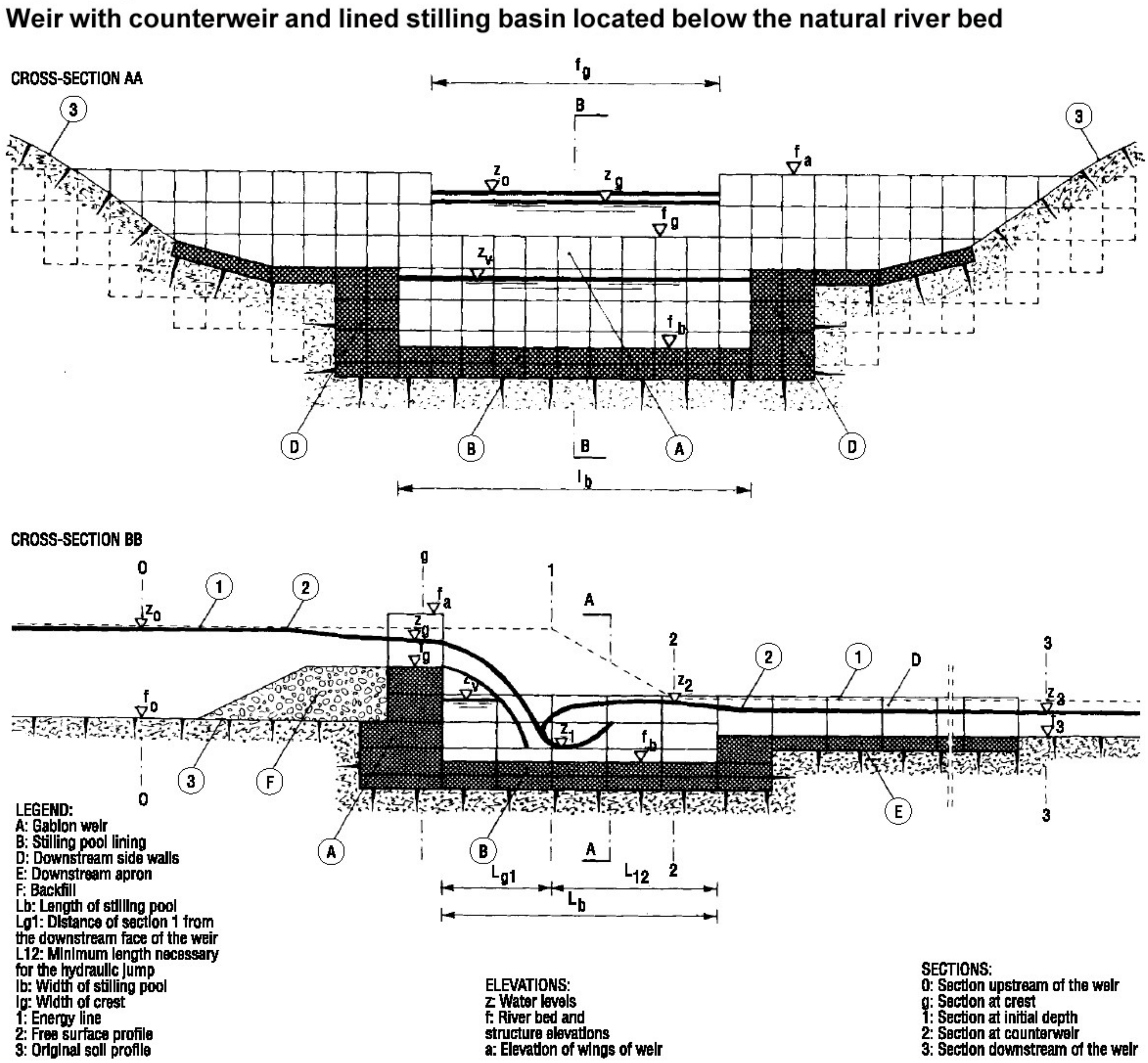


Fig. 3.19 - Weir with counterweir and lined stilling basin located below the natural river bed

Stepped weirs

This kind of weir is generally used in relation to low specific flows and significant drop heights. Experimental observations, conducted by Cemagref, show that stepped weirs are particularly convenient for specific flows inferior to 3 $m^3/s/m$. For higher values of the specific flow, the gabions step could be damaged. In stepped weirs, the energy dissipation takes place already on the steps. In fact, the experimental evidence shows a 10%-30% diminution of the length of the stilling basin compared to the length obtained using traditional methods (Peyras , Royet , Degoutte. 1991).

With reference to a weir's specific flow and downstream side slope, four different kinds of hydraulic situations can take place:

- nappe flow, with flow alternatively in subcritical and supercritical range,
- nappe flow, with flow always in supercritical range,
- partial nappe flow, with flow always in supercritical range,
- skimming flow.

Battered weirs

This kind of weirs is generally well suited to significant drop heights and low specific flows (inferior to 1 $m^3/s/m$). The specific flow has to be limited only if the weirs are built with gabions, which can easily be damaged. In fact, violent water flows can provoke stones' rubbing within gabions baskets, consequently leading to stone or net breakage. Transported materials colliding with gabions can also provoke net tear.

Battered weirs will require a reinforced concrete lining if the specific flow is higher than 1 $m^3/s/m$. A detailed explanation of the methods used to dimension the stilling basin in the case of battered weirs can be found in U.S. Bureau of Reclamation 1987.

Verification against piping failure

In presence of a weir, the water level builds up bringing about a difference in level between the upstream and downstream side of the weir. The gradient thus established gives rise to a ***seepage reticule*** underneath the structure. The seepage's characteristics largely depend on the soil's materials. Given that gabions structures are generally built on pervious soils, the seepage phenomenon can provoke the formation of springs downstream the structure. A substantial spring flow can transport particles of soil material, progressively increasing water seepage and the amount of material transported, eventually leading to the ***structure's failure***.

To test the weir against the possibility of a piping failure, the seepage reticule has to be determined. This will allow us to identify the seepage flow path and hydraulic gradient in the area underneath the structure (see figure 3.20). In order to determine the flow path and the hydraulic gradient of the seepage reticule, we will have to solve a composite system of differential equations. However, for the small structures dealt with in this work, the test against seepage can be generally accomplished using the ***Bligh method*** (U.S. Department of the Interior Bureau of Reclamation. 1987, Maccaferri 1990a). According to this method, the structure will be tested against seepage when the following relation is verified:

$$L > c \times \Delta h$$

whereby:

- L seepage path below the structure (the length of vertical is tripled in the sum),
- c coefficient depending on soil characteristics (see figure 3.21 for its values),
- Δh water level difference between upstream and downstream the weir.

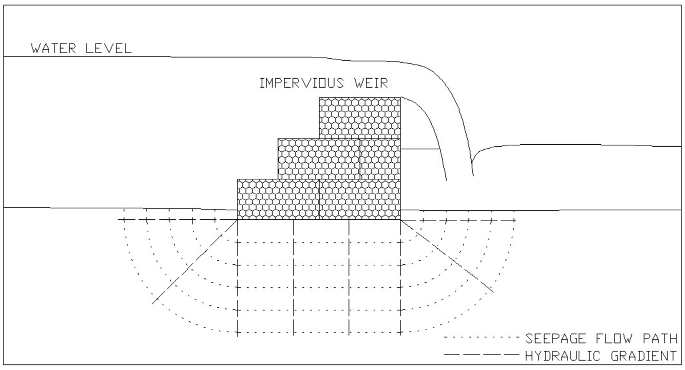


Fig. 3.20 – Seepage reticule below a weir

Bligh coefficient (c) values

Type of soil	c	Size of particle (mm)
Fine silt and mud	20	0,01 – 0,05
Coarse silt and very fine sand	18	0,10
Fine sand	15	0,12 – 0,25
Medium sand	12	0,30 – 0,50
Coarse sand	10	0,60 – 1,00
Gravel	9 - 4	> 2,00
Hard clay	6 - 3	0,005

Fig. 3.21 – Bligh coefficient (c) values

If the above mentioned relation is not verified, then the weir section will have to be modified. A lengthening of the seepage path can be achieved in two ways (see figure 3.22):

- build a gabions' apron downstream the weir (case A)
- build an impervious cut-off below the structure (case B)

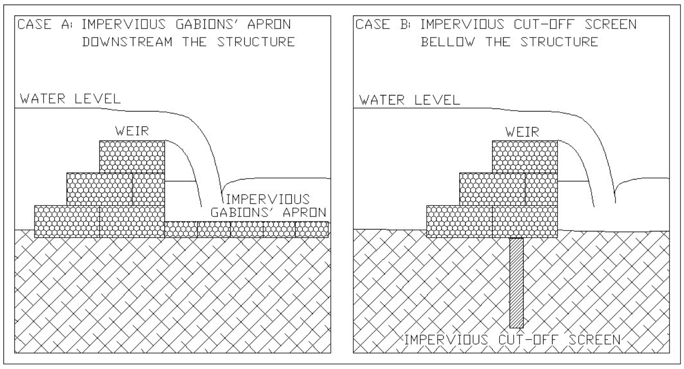


Fig. 3.22 - Methods for lengthening seepage paths (case A and B)

The actual seepage reticule will correspond to the one designed in figure 3.20 only in the hypothesis of a completely impervious structure, otherwise the seepage reticule will be influenced by the high permeability of gabions, which attract the seepage, causing it to deviate its path, as shown in figure 3.23.

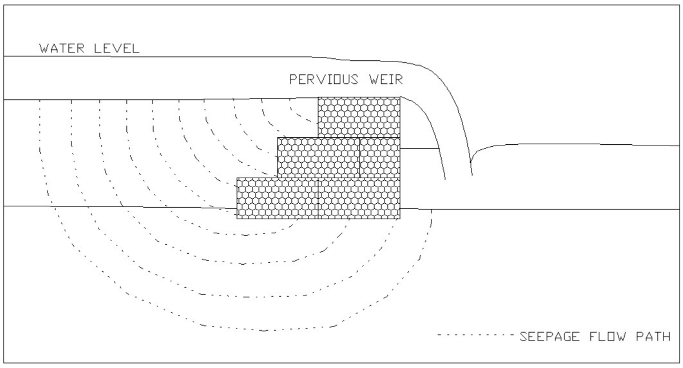


Fig. 3.23 - Seepage reticule in the hypothesis of pervious gabions

In these hypothesis, the probability that soil particles are transported by seepage is high. In fact, the soil in contact with gabions is exposed to water seepage pressure but, on the other hand, it is also influenced by atmospheric pressure or water pressure below the water level. However, water's seepage pressure is generally higher than the latter two, and it can remove fragments of material in contact with gabions. To eliminate this problem, a layer of graded material, such as gravel, should be interposed between gabions and soil. Also a layer of geotextile, generally easier to install, would suit the purpose. Another feasible solution consists in placing an impervious membrane between gabions and soil. Through these devices the weir is rendered completely impervious and Bligh's theory can be used for testing the weir against excessive seepage.

Whether graded material or geotextile is chosen, the layer will inevitably be obstructed by the particles transported by water's seepage and will consequently become impervious.

3.3.4 - Stability analysis

- *Loads analysis*
- *Horizontal thrusts*
- *Vertical loads*
- *Test against overturning*
- *Test against sliding*
- *Verification against uplifting*
- *Resistance test for the foundation soil*

This paragraph illustrates the procedure used to test the stability of gabions structures (Maccaferri 1990a). First of all, it will be useful to introduce loads analysis, with regard to both horizontal thrusts and vertical loads on the structure. Then, four different stability tests for gabions structures will be explained, namely:

- against overturning,
- against sliding,

- against uplift,
- against excessive pressure on foundation soil.

Loads analysis

With respect to figure 3.24, loads on the weir structure are explained below.

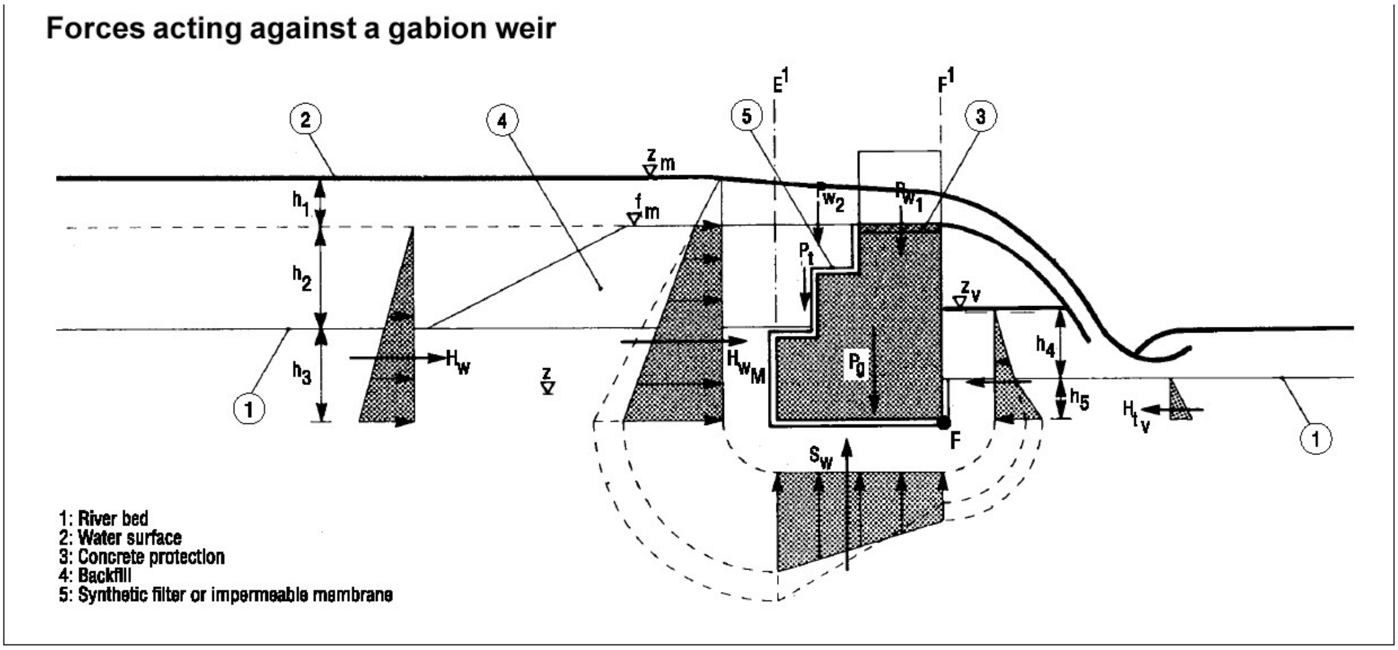


Fig. 3.24 - Weir cross-section for stability analysis

Horizontal thrusts

- water:

$H_{wm} = 0.5 \times \gamma_w (2 \times h_1 + h_2 + h_3) \times (h_4 + h_5)$ on the upstream side

$H_{wv} = 0.5 \times \gamma_w \times (h_4 + h_5)^2$ on the downstream side

- soils

$H_{tm} = 0.5 \times \gamma_{tw} \times \lambda_a \times (h_2 + h_3)^2$ on the upstream side

$H_{tv} = 0.5 \times \gamma_{tw} \times \lambda_a \times h_5^2$ on the downstream side

Vertical loads

- water

$P_{w1} = S_{w1} \times \gamma_w$

$P_{w2} = S_{w2} \times \gamma_w$

- soil

$P_t = S_{soil} \times \gamma_{t1}$

- water uplift

$S_w = \gamma_w \times b \times (h_4 + h_5) + 0.5 \times \gamma_w \times b \times (h_1 + h_2 + h_3) - (h_4 + h_5)$

- structure's weight

$P_g = S_{sub.\ struc.} x\ \gamma_{g1} + S_{dry\ struc.}\ x\ \gamma_g$

In these relations, symbols represent:

γ_w	water unit weight (it generally varies between 1000 and 1100 kg/m^3, but it can assume higher values in case of a very important solid transportation),
γ_g	gabion unit weight ($\gamma_g = \gamma_s$ x (1 – n_g)),
γ_s	material unit weight (see figure 3. for the values),
n_g	gabions porosity (generally about 0.3),
γ_{g1}	gabions saturated unit weight ($\gamma_{g1} = \gamma_s$ x (1 – n_g) + n_g x γ_w),
γ_{tw}	soil submerged unit weight (γ_{tw} = (γ_s - γ_w) x (1 – n)),
n	soil porosity,
γ_{t1}	soil saturated unit weight ($\gamma_{t1} = \gamma_s$ x (1 – n) + n x γ_w),
λ_a	coefficient of active earth pressure ($\lambda_a = tg^2$ (45 - φ / 2)),
φ	soil angle of friction.

Rock specific weight

Type of rock	Unit weight Kg/m^3
Basalt	2900
Granite	2600
Trachyte	2500
Tuff	1700
Soft limestone	2200
Hard limestone	2600
Sandstone	2300

Fig. 3.25 - Soil unit weight

Test against overturning

The test must be conducted in relation to the structure's overturning around the F point (see figure 3.24). Below are detailed overturning and stabilising forces:

overturning forces

- horizontal thrusts by water (H_{wm} , H_{wv}),
- horizontal thrusts by soil (H_{tm}),
- water uplift (S_w)

stabilising forces

- structure weight (P_g),
- water weight (P_{w1}, P_{w2}),
- soil weight (P_t),
- horizontal thrusts on the downstream side by water and soil (H_{wv} , H_{tv}).

Multiplying the forces for their respective arms and summing all the overturning and stabilising moments, we obtain the following relation, which shows the structure's stability coefficient:

$s_r = M_s / M_r$ coefficient against overturning

where M_s is the sum of the stabilising moments and M_r is the sum of overturning forces. For small structures $s_r > 1.3$. For more important structures, instead, the stability coefficient against overturning will take up higher values.

Other tests against overturning are normally required at different structure levels, but the procedure illustrated above is generally the fundamental one, for the majority of structures.

Test against sliding

To carry out this test, horizontal (Σ H) and vertical (Σ V) forces' resultants must be calculated. The following relation has to be verified:

$\Sigma H < tg\, \varphi \times \Sigma H$

whereby φ represents the friction angle between gabions and foundation soil. A common value for the friction angle φ is 35° with a corresponding tg $\varphi \cong 0.7$. In this case, the stability coefficient against sliding will be expressed as:

$s_s = tg\, \varphi \times \Sigma H / \Sigma H$

As for the overturning, s_s must be greater than 1.3 for small structures. For more important structures, the stability coefficient against sliding will take up higher values. suitable.

Verification against uplifting

Lining the stilling pool is usually necessary to protect against seepage failures. Where this lining is constructed using gabions or mattresses laying on a reverse filter or a geotextile, it is necessary to check the stability of the lining against hydraulic uplift, and check that the uplift force due to seepage water is not greater than the combined weight of the lining and of the interstitial water, filters, and the water passing over the lining.

It is therefore necessary to evaluate the distribution of pressures under the stilling pool by drawing a flow diagram or by using the simplified method already suggested. With reference to figure 5.24, the pressure, p, at each point of the foundation is

$$p = \gamma_w x \left[\left(z_0 - \frac{z_0 - z_3}{L_f} \right) - z_x \right]$$

If h is the depth of the water above the apron and s the thickness of the apron, then the coefficient of stability against uplift is

$$S_g = \frac{(\gamma_{gl} x s) + (\gamma_w x h)}{p}$$

Acceptable values of safety factor S g are between 1.1 and 1.2.

Resistance of the foundation soil

For each section under examination, all the forces H wM , H wV , H tM , H tV , P w1 , P w2 , t P, P g and S w of Figure 5.17 are computed for the worst case. The intensity and the trend of the resultant R of the acting forces, its inclination and the centre of gravity, are then determined. It is conservatively assumed that the gabion foundation surface remains flat and that the Foundation soils much less rigid than the gabion structure. With regard to this second assumption, the results of experiments indicate that the rigidity of gabions is comparable to that of soil. If the centre of gravity of X is within the middle third – MN – the pressure is distributed over the whole foundation, and the maximum pressure, . B , at the downstream toe, B, in kg/cm 2 , is found from:

$$\sigma_b = 6x \frac{V x \overline{XM}}{100 x \overline{\overline{AB}}^2}$$

where:

V is the vertical component of the resultant R (kg); and XMand AB are distances (cm).

If the centre of gravity is coincident with the extreme edge of the middle third (N), the maximum pressure, σ_b , is:

$$\sigma_b = 2x \frac{V}{100 x \overline{\overline{AB}}}$$

A centre of gravity outside the middle third - MN - is to be avoided, since, in accordance with the assumption made above, only part of the foundation is utilized. In practice this is an unlikely situation in a gabion structure due to its great flexibility, but in such a case, the pressure σ_b would be:

$$\sigma_b = 2x\frac{V}{100x3x\overline{XB}}$$

The maximum pressure σ_b should be lower than the foundation soil bearing capacity given for various soils in Table 5.7.

Indicative bearing capacities of soils

Type of soil	Bearing capacity Kg/m^2
1 Uncompacted, borrow soil	0 - 1
2 Compacted cohesionless soils	
a) sand, grain diameter <1 mm	2
b) sand, grain diameter 1-3 mm	3
c) sand and gravel (at least 1/3 gravel)	4
3 Cohesive soils, classed by to water content	
a) fluid; plastic fluid	0,0
b) plastic soft	0,4
c) plastic solid	0,8
d) semi-solid	1,5
e) solid	3.0
4 Rock in good condition (if fissured or liable to disaggregation, the indicated bearing capacitymust be reduced to less than half)	10 - 15

Table 5.7