Flexible gabion structures in river and stream training works

Section one
Weirs for river training and water supply

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Preface

The growth of interest shown by civil engineers in the use of gabions for the building of flexible structures since the mid 1960’s has not only led to an increase in the number of installations but also to more sophistication in design and to a greater variety of applications. In consequence the demand for technical information and data has risen accordingly.

In answer to this demand, Officine Maccaferri S.p.A. who produced the first factory made gabion unit, are currently preparing and publishing a series of handbooks dealing with the design and construction of some of the applications requiring relatively complex design procedures. "Flexible linings of Canals and Canalised Water Courses" has been in print for sometime and is now followed by this publication on the subject of weirs, falls and spillways.

Acknowledgement must be made to Dr. Eng. Raffaele Agostini and Dr. Eng. Maurizio Masetti of Officine Maccaferri S.p.A., who were responsible for the preparation, and to Dr. Eng. Alberto Bizzarri, Professor of Soil Conservation in the Faculty of Engineering of the University of Bologna for his invaluable assistance.

Bologna, May 1981

Dr. Eng. Andrea Papetti
GENERAL MANAGER
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The object of this brochure is to suggest methods and outline simple criteria for the design and construction of weirs in gabions and Reno Mattress.

Principally it deals with small and medium size weirs of up to twenty metres high founded on soils whose main characteristics are limited bearing and shear strengths. On such soils the flexible gabion weirs work well compared to rigid structures which are likely to fracture if settlement takes place. It is not our intention to put forward new and original design theories, but to assist consultants and contractors who have chosen gabions in preference to other materials. For a more detailed study of the subject the reader should refer to the technical publications listed in the bibliography. Officine Maccaferri S.p.A. will always be at the disposal of engineers who are interested in the use of gabions and Reno Mattress and who require assistance in the solution of particular problems.
CHAPTER I

Gabion weirs

1 Preamble

In the following pages a variety of gabion weirs are described and discussed. Their purposes include water diversion (for irrigation, domestic water supply and industrial requirements), river training and the formation of reservoirs.

Where the river bed consists of fine sand and silts which are permeable, easily eroded, and have low bearing capabilities, the gabion structure offers a more convenient solution than most other materials. In these soils, where the height of the crest above downstream bed level does not exceed 5 metres, the vertical faced weir is the best choice.

In the same conditions, for structures 5 to 20 metres high, a sloped weir is preferable as the stability is improved and the absence of the free fall guarantees a better control of the energy of the water. To prevent the migration of the stone fill in the gabions, the faces of the crest, slope and stilling pool are usually grouted with sand asphalt mastic or concrete.

Downstream bed protection by means of a counterweir and stilling pool and upstream by an apron must be included in the design of structures based on silts and sands, but can sometimes be omitted in the case of stable soils where the vertical faced type can occasionally be considered for heights greater than 5 metres.

2 The field of application of gabion weirs

As mentioned above, gabions are used for the construction of weirs for a wide variety of purposes including diversion works, but since these usually form part of complex schemes they are not specifically treated in detail in this brochure. General information on them may be found in the references given in the bibliography, [1], [2], [3], [4], [5], [6], [7], [8], [9], [10], [11], [12], [13], [14]. Gabion weirs are more frequently found in river training, soil stabilisation, and water supply schemes. In addition to flexibility which allows them to deform while remaining structurally sound, other advantages which gabions offer are relatively low cost and simplicity of construction. On many sites stone is locally available, which means that the only material requiring to be transported any distance to the site is the gabions themselves. Sophisticated plant and equipment are unnecessary. Ordinary standard front loaders or cranes fitted with dragline buckets are usually used for filling. Skilled craftsmen are not needed, and labour can be fully trained within a matter of days to carry out assembly and erection of the units in the required manner. Of the purposes mentioned above, the one in which gabions are used more than any other is river training. The brochure has been written with this particularly in mind and accordingly the subject is treated in detail in the following chapter.
2.1 The use of gabion weirs in river training

In mountainous countries the control of erosion in torrents and streams can be of major importance. Where proper control is maintained, the whole area of the stabilised river basin benefits, since the halting of bed degradation in the upper reaches reduces both the occurrence of landslides, and the deposition of material in the lower reaches. In the latter, examples of the benefits are reductions of flood risk, of silting of reservoirs and canals and of navigation costs (figs. 1, 2).

Erosion in streams is checked by lowering the velocity of water to a value at which it ceases to move the soil particles forming the bed and banks. This is achieved by reducing the gradient to obtain a stable velocity and hence equilibrium and in practice such conditions are attained by the construction of a series of weirs, or check dams, so that the slope between the toes and the crests is stable for the soil concerned, the excess water energy being dissipated at the toe of each structure.

The design of a scheme of this kind must obviously start with the determination of the stable slope [15], [16] which can be calculated using formula (1) below, provided that it is applied to reaches that do not have abrupt contractions, i.e. where the flow can be considered as uniform [15].

\[ i_e = \frac{(vu)^{10/3} B^{4/3} n^2}{Q^{4/3}} \]

where:

- \( i_e \) (m/sec) : stable slope
- \( u_e \) (m/sec) : maximum permissible velocity, depending on the size of bed materials, at which the erosion of river bed starts. The suggested values of \( u_e \) for different types of bed materials, are shown in table 1 [17], [18], [19];
- \( v \) : ratio between the mean velocity of water and the corresponding velocity at the river bottom: this ratio is nearly equal to 1.3-1.5;
- \( B \) (m) : wetted perimeter, which can be generally considered equal to the width of the river;
- \( n \) (m\(^{-1/3}\) sec) : coefficient of roughness of the river, according to the Manning formula \( \chi = \frac{1}{n} R^{1/6} \), where \( \chi \) is the resistance coefficient of uniform flow and \( R \) the hydraulic radius. In table 2 the values of a few natural watercourses extracted from Ven Te Chow [20] are shown;
- \( Q \) (cumecs) : flood discharge according to which the river training is designed; generally discharges having return periods of say 20 to 30 years are considered for small unimportant structures. For larger important works, 50 to 100 years return periods are taken.
When the above parameters along the whole length of the reach in question are known the stable slope is calculated, after which the position and height of the structures must be determined.

If a stretch of a river, having natural slope $i$, is to be trained to a slope $i_e$ by means of a series of weirs at equidistant points (fig. 3), the height $H$ and distance $l$ between two weirs are connected by the relation: $H = H_1 - H_2 = (i - i_e)l$. Consequently, the number $n$ of weirs necessary for the training of the considered length $L$, is:

$$\frac{L}{l} = \frac{L(i - i_e)}{H}$$

(2)

In general, it is preferable to build small and closely separated structures instead of high ones, particularly where the soil is subject to erosion, in order to disturb the natural watercourse as little as possible.

As an example, figs. 4, 5 illustrate the corrected profile of a torrent.

Recently, in addition to traditional forms of weirs constructed for the prevention of river bed erosion, other types of structures have been developed for this particular purpose amongst others but having different characteristics.

![Diagram showing a method of computing the height and spacing of check dams for bed stabilization.](image)

Table No. 1 - Maximum permissible velocities recommended by Fortier and Scobey (for straight channels of small slope, after aging) [17], [18], [19].

<table>
<thead>
<tr>
<th>Material</th>
<th>Clear water V (m/sec)</th>
<th>Water transporting colloidal silts V (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine sand, colloidal</td>
<td>0.45</td>
<td>0.76</td>
</tr>
<tr>
<td>Sandy loam, noncolloidal</td>
<td>0.53</td>
<td>0.76</td>
</tr>
<tr>
<td>Silt loam, noncolloidal</td>
<td>0.60</td>
<td>0.91</td>
</tr>
<tr>
<td>Alluvial silts, noncolloidal</td>
<td>0.60</td>
<td>1.06</td>
</tr>
<tr>
<td>Ordinary firm loam</td>
<td>0.76</td>
<td>1.06</td>
</tr>
<tr>
<td>Volcanic ash</td>
<td>0.76</td>
<td>1.06</td>
</tr>
<tr>
<td>Stiff clay, very colloidal</td>
<td>1.14</td>
<td>1.52</td>
</tr>
<tr>
<td>Alluvial silts, colloidal</td>
<td>1.14</td>
<td>1.52</td>
</tr>
<tr>
<td>Shales and hardpans</td>
<td>1.82</td>
<td>1.82</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>0.76</td>
<td>1.52</td>
</tr>
<tr>
<td>Graded loam to cobbles when noncolloidal</td>
<td>1.14</td>
<td>1.52</td>
</tr>
<tr>
<td>Graded silts to cobbles when colloidal</td>
<td>1.22</td>
<td>1.67</td>
</tr>
<tr>
<td>Coarse gravel, noncolloidal</td>
<td>1.22</td>
<td>1.82</td>
</tr>
<tr>
<td>Cobbles and shingles</td>
<td>1.52</td>
<td>1.67</td>
</tr>
</tbody>
</table>

(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).
Table No. 2 - Values of the roughness coefficient "n" for natural streams [20].

<table>
<thead>
<tr>
<th>MIN.</th>
<th>NORM.</th>
<th>MAX.</th>
</tr>
</thead>
</table>

1. **Minor streams** (top width at flood stage < 100 ft)
   - **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 and shoals 0.033 0.040 0.045
     4. Same as above, but some weeds and stones 0.035 0.045 0.050
     5. Same as above, lower stages, more ineffective slopes and sections 0.040 0.048 0.055
     6. Same as 4, but more stones 0.045 0.050 0.060
     7. Sluggish reaches, weedy, deep pools 0.050 0.070 0.080
     8. 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

2. **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 winter 0.035 0.050 0.060
     3. Light brush and trees, in summer 0.040 0.060 0.080
     4. Medium to dense brush, in winter 0.045 0.070 0.110
     5. Medium to dense brush, in summer 0.070 0.100 0.160
   - **d) Trees**
     1. Dense willows, summer, 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

3. **Major streams** (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance
   - **a) Regular section with no boulders or brush** 0.025 — 0.060
   - **b) Irregular and rough section** 0.035 — 0.100
Among these are "frame" weirs, "filtering weirs", "selective weirs", etc. [21], [22], [23], [24].

While traditional weirs stop all the bed load and most of the suspended material, and finish their capacity of retention once the channel upstream has filled to the level of the cill, the new types have a continuous retaining capacity. Their shape enables them to pass alluvium during low discharges, but stop large sized objects such as boulders and logs which could cause damage downstream. Hence, they can select the size of material to be retained and to be passed. The finest material is allowed to flow downstreams to maintain equilibrium, which is particularly important in the middle reaches of river.

Gabion weirs on the other hand offer a different advantage. The structure can be changed in height and size simply by building up or removing courses of gabions on the existing structure. This can be very convenient when control works are required urgently on rivers on which the collected hydrological information is meagre. After a period of operation, the shape of the structure can be adjusted according to requirements, and progressive adjustments can be made thereafter (figs. 6, 7).

An example of the re-shaping of a gabion weir is shown in fig. 8. In the first stage the structure had a stepped face and functioned for about ten years as a check dam. In the second stage it was reconstructed as a sloped structure, gabions being added to the original stepped core. The whole structure was afterwards sealed with sand asphalt mastic, to become a dam retaining a reservoir.

At the conclusion of this section on River Training it must be mentioned that weirs are also employed to protect structures such as bridges and for raising the level for intakes (figs. 9, 10).
1: Existing gabion weir (1st phase)
2: Undermined apron to be removed
3: Re-shaping (2nd phase)
4: Backfill
5: Mastic grouted Reno Mattresses
6: Permeable filter
7: Concrete cut-off

Fig. 8 - LIBYA - Bengazi - The re-shaping of a gabion weir after the original structure was undermined. The stepped downstream face was modified to a smooth glacis.

Fig. 9 - ITALY - Carturo (Padova) - Weir on the River Brenta.

Fig. 10 - Diagram illustrating a sloped weir.
Gabion weirs are classified in three types, according to the shape of their downstream face at the centre of flow:
- vertical weirs,
- stepped weirs,
- sloped weirs.

The vertical weir is certainly the simplest type, and is often used for small weirs in a system controlling a reach of a stream.

An example of vertical gabion weir is shown in figs. 11, 12.

In the vertical gabion weir, the nappe is not only aerated, but separated from the downstream face. Since this means that the weir mesh is protected against abrasion and impact by heavy bed material carried in spate conditions, it is a type recommended for training works on mountain torrents. The only mesh which is exposed to abrasion is the crest, which must be protected. Suitable materials are: timber or steel sheets securely fastened to the wire netting, or concrete capping, with joints at say 2 metres centres, cast in situ after the structure has settled (chap. II, par. 2.2).

An essential component of the weir, is the upstream ramp (fig. 12), preferably formed of compacted clay, but otherwise of any suitable locally available material. The ramp serves at least three purposes: protection of the upstream side of the gabion against damage, presentation of a smoother profile to the flow and added stability to the structure [17].

Fig. 11 - U.S.A. - Gabion vertical weir.
In the design of vertical weirs, maximum attention must be paid to the dissipation of the kinetic energy of the cascade at the toe of the structure. In certain cases, the cascade is allowed to scour the bed and form a pool where the energy is dissipated in the cushion of water and in the formation of a hydraulic jump. A secondary weir is placed at the downstream end of the pool to control the formation of the jump and to restrict the extension of the pool downstream. The latter is necessary to ensure that the main structure is not undermined (fig. 13).

In the case of high weirs in erodible soils, the river bed in the stilling pool is protected by an apron, and the control of the hydraulic jump is obtained by means of:
- a broad crested counterweir: to ensure that the tailwater does not affect the conditions of flow in the pool;
- an abrupt rise in the other case (fig. 14).
The stepped weir has no essential differences from the vertical type, but the water flowing over the weir dissipates a part of its energy at each step (figs. 15, 16).

This type of structure should only be used for small weirs where the values of discharge for unit width are limited; in any case it should be avoided where a heavy bed load is carried which could cause damage to the mesh on the steps.

For large weirs, and when the height of the structure ranges from 10 to 15 metres, the requirements of greater stability and improved hydraulic behaviour dictate the use of weirs with a sloping downstream face.

The slope of the face must be designed so that the nappe adheres to it. At the toe of the slope, a stilling pool formed by a broad crested weir or abrupt rise will be necessary (see figs. 9, 10).
Sloped weirs are generally found in training schemes on major rivers carrying light bed loads and in which the soils are of fine granular material having a poor bearing capacity. Where a heavy bed load of shingle and boulders is carried, the crest, slope and stilling pool must be protected from abrasion by concrete or asphaltic mastic lining.

In the following chapters, design criteria and suggested methods of construction are set out for each of the types of weirs classified above.
CHAPTER II
Design criteria for vertical and stepped weirs. Construction details

1 General design criteria

Hydraulic and structural stability criteria. Hydraulic calculations involve: the design of the crest, through which the maximum discharge is maintained in the centre of the river; the design of the stilling pool, for energy dissipation and control of scour downstream of the structure; and the control of seepage under and around the weir, in order to avoid washing away of the finest particles of soil.

Structural stability calculations involve checking: the stability of the weir and counter-weir against overturning and sliding; the stability of the bed of the stilling pool against uplift; and the bearing pressures on the structure and the foundation soil.

All computations are made for a section through the centre of the crest, which is generally the "worst" section.

With regard to their hydraulic behaviour, there are three fundamental types of weirs. An example of each is shown in figs. 17, 18, 19 respectively.

The most simple and commonly used type is that in fig. 17, i.e., a gabion weir with a counter-weir placed at a suitable distance downstream. There is no lining to the stilling basin, and the nappe erodes the soil to form a pool deep enough to dissipate the energy of the water. The river bed is generally left unprotected upstream of the main weir and downstream of the secondary weir.

In the vertical weir illustrated in fig. 18, the bed of the stilling pool is protected and the hydraulic jump is controlled by a broad-crested counter-weir. The river bed may be protected upstream and downstream of the structure as well. In this type, the critical state obtained on the secondary weir prevents the behaviour of the flow in the stilling pool from being affected by the flow conditions downstream.

In the vertical weir shown in fig. 19, the apron protecting the stilling pool is below original river bed level, and the jump control is obtained by an abrupt rise. Here again the bed may be protected upstream and downstream of the structure. The depth of the stilling pool below the original river bed is established in such a way as to ensure that the jump is drowned, when the flow is subcritical.
Fig. 17 - Gabion vertical weir and counterweir; stilling pool with unlined floor.
LEGEND:
1: Energy line
2: Free surface profile
3: Original soil profile
4: Profile of maximum bed scour
A: Gabion weir
B: Backfill
C: Counterweir
L: Width of crest

ELEVATIONS:
z: Water levels
f: River bed and structure elevations
a: Elevation of wings of weir

SECTIONS:
0: Section upstream of the weir
1: Section at max. bed scour
2: Section at sequent depth
3: Section downstream of the weir
Fig. 18 - Gabion vertical weir with lined stilling pool. Jump control by broad crested weir.
CROSS-SECTION AA

CROSS-SECTION BB

LEGEND:
A: Gabion weir
B: Stilling pool lining
C: Counterweir
D: Downstream side wall
E: Backfill
F: Downstream apron
L: Length of stilling pool
L₁: Distance of section 1 from the downstream face of the weir
L₂: Minimum length necessary for the hydraulic jump
W: Width of stilling pool
1: Energy line
2: Free surface profile
3: Original soil profile

ELEVATIONS:
z: Water levels
f: River bed and structure elevations
a: Elevation of wings of weir

SECTIONS:
0: Section upstream of the weir
g: Section at crest
1: Section at initial depth
2: Section at sequent depth
c: Section at counterweir
3: Section downstream of the weir
Fig. 19 - Gabion vertical weir with lined stilling pool. Jump control by abrupt rise.
LEGEND:
A: Gabion weir
B: Stilling pool lining
D: Downstream side walls
E: Downstream apron
F: Backfill
L: Length of stilling pool
L': Minimum length necessary for the hydraulic jump
L'': Distance of section 1 from the downstream face of the weir
l: Width of stilling pool
l': Width of crest
1: Energy line
2: Free surface profile
3: Original soil profile

ELEVATIONS:
z: Water levels
f: River bed and structure elevations
a: Elevation of wings of weir

SECTIONS:
0: Section upstream of the weir
g: Section at crest
1: Section at initial depth
2: Section at sequent depth
3: Section downstream of the weir
2 Crest design. Construction details

2.1 Crest design

The crest is that central part of the weir through which the maximum discharge flows.

In all of the three cases considered above, the crest must be designed so as to maintain the design discharge in the centre. This will prevent over-topping of the wings and scouring of the banks, and also any backing-up behind the weir. With reference to the symbols of figs. 17, 18, 19 a rectangular crest can be designed according to the equation:

\[ Q = \mu l_p(z_0 - f_p)\sqrt{2g(z_0 - f_p)} \]

where:

- \( Q \) (cumecs) : rate of discharge assumed for the design. The flood discharge corresponding to a certain “return period” is taken for the calculation, considering the degree of risk which is acceptable in relation to the importance of the structure. For instance, in small weirs employed in the training of mountain streams, it is sufficient to consider a return period of say 10 to 20 years, because the importance of the structure is modest and the damage caused by a possible underdesign or failure is limited. For larger weirs used for water diversion or protection of structures, or when built-up areas are in the vicinity of the river, longer return periods, and consequently greater flood discharges, must be taken into consideration;
- \( \mu \) : discharge coefficient, varying from 0.385 to about 0.6. The discharge over the crest may be taken as for an unsubmerged broad crested weir. Where the upstream velocity head is negligible, and consequently the total head approximates the piezometric head, a discharge coefficient \( \mu = 0.385 \) is used. Where it is necessary to take into consideration the influence of the upstream velocity head the coefficient \( \mu \) takes a higher value, in relation to the higher approach velocity. When choosing the value of \( \mu \) the effect of possible lateral contraction should also be considered. Finally, when the crest is submerged, it is necessary to take into account the influence of the downstream head of water;
- \( g \) (m/sec²) : acceleration due to gravity;
- \( l_p \) (m) : width of the crest;
- \( z_0 \) (m) : elevation of the water surface above datum, measured upstream from the crest, at a distance where the effects of surface contraction have no influence;
- \( f_p \) (m) : elevation of the crest, above the datum.

Usually, \( l_p \) must satisfy the necessity to maintain the discharge in the centre of the water course, in order to avoid bank scour. Elevation \( z_0 \) is fixed so as to avoid a dangerous backwater effect upstream of the weir, during spate conditions, and to ensure a sufficient freeboard against flooding, without requiring costly embankments. Elevation \( f_p \) is determined by the weir’s function; for instance, if the weir is to be built for the creation of a reservoir, \( f_p \) is equal to the maximum water level in the reservoir; if the weir is part of a river training scheme, \( f_p \) represents the elevation of the new profile to be obtained.

By the use of equation (3), the most suitable values of \( l_p, z_0 \) and \( f_p \) can be determined, when the discharge \( Q \) is known and the coefficient \( \mu \) has been estimated.

To simplify calculation, \((z_0 - f_p)\) has been plotted against \( \mu \) and \( q \) in fig. 32, page 32.

With \( z_0 \) and \( f_p \) computed, the height of water \((z_0 - f_p)\) above the crest may be determined, as it is generally equal to 2/3 of the upstream total head above crest level \( f_p \). Finally, the level of the top of the wings of the weir is made 30-40 cm higher than \( z_0 \).

When the crest is not rectangular, but trapezoidal (as in figs. 20, 21) or curved (as in figs. 22, 23) or otherwise shaped, the relation between the discharge and the head of water can be obtained by studying the critical state on the crest; each value of the critical depth \((z_0 - f_p)\) on the crest corresponds to a value of the critical velocity \(v_c = \sqrt{\frac{g\Omega}{b}}\) and discharge

\[ Q = v_c\Omega = \Omega \sqrt{\frac{g\Omega}{b}} \]

where:

- \( g \) (m/sec²) : acceleration due to gravity;
- \( \Omega \) (m²) : area of cross section of flow for a depth equal to the critical depth;
- \( b \) (m) : corresponding width of water surface.

Having found the relation between the cross section and the discharge for the flow at the critical state, it is possible to obtain the critical depth and the specific head on the crest (equal to the upstream total head, ignoring any energy losses) corresponding to the design discharge.
2.2 Construction details

It will be useful at this point to give some details about construction, with particular attention being paid to the protection of the crest against abrasion and corrosion.

Pedestrians and animals crossing the structure do not cause any appreciable damage to the mesh, but it can be attacked by chemical corrosion or by the abrasion and impact of heavy bed load material transported by the river in spate.

The steel wire is protected against rust by a heavy zinc coating. Where the water is heavily polluted the special PVC sheathed galvanised wire should be used.

In any case, even if some rust occurs after many years of service, the structure need not lose its effectiveness, as additional protection can be given or the mesh replaced. In practice soil fills the voids between the stones in time, causing a cementing of the fill material, and vegetation will tend to key the structure to its surroundings.
An example of the durability of a gabion structure, even after all the zinc coating has disappeared, is shown in figs. 24, 25. Fig. 24 shows a weir immediately after construction (in 1930), and in fig. 25 the same weir 35 years later. The weir, now covered by vegetation, has become part of the river itself.

As indicated in the previous chapter at paragraph 3, if the water course carries large quantities of heavy bed load, it is necessary to protect the mesh on the crest. Generally, a timber protection is the easiest and cheapest form, assuming it is available on site and this protection may be carried out as soon as the main structure is completed (figs. 26, 27).
Concrete is however the most commonly used protection. As the rigidity of concrete is not consistent with the flexibility of the gabion structure, which is also subject to settlement, it is necessary to provide a number of joints in the concrete skin. Moreover, it is best to form this concrete skin some months after construction of the gabion weir, i.e., after most of the settlement has taken place.

In figs. 28, 29 a reinforced concrete protection is shown. Occasionally, the mesh may be protected with steel sheets. The methods of protection described above can be applied both to vertical and stepped weirs.

3 Stilling pool design: construction details

In chapter I, paragraph 3, a brief description of the three principal types of vertical gabion weirs was given. The difference between them is only the way in which the energy of the water is dissipated in the stilling pool. The criteria for the design of the three types of stilling pool are stated below.

3.1 Stilling pool with unlined floor

(See scheme in fig. 17). In river training structures, when the head of water to be dissipated does not exceed a few metres, and the river bed is formed of coarse or very compacted material, protection to the bed of the stilling pool can be omitted.

Where however the river bed is formed of loose material, maximum care is necessary in the evaluation of the greatest depth of scour, caused by the clear fall, in which a natural pool is formed where the water is able to dissipate its energy. In this case, the foundations of the weir should be deeper than this maximum possible scour of the river bed, in order to avoid undermining the structure. Under the fall, it is wise to place random stones of such a weight that the stream cannot wash them away.
For the stilling pool design, it is necessary to evaluate both the maximum distance of the free fall from the structure, and the depth of scour (Fig. 30).

The first problem involves the study of the trajectory of a body falling, in the absence of friction, from an elevation \((z_g - f_g)\), and with a horizontal velocity approximately equal to the critical velocity \(\sqrt{g(z_g - f_g)}\).

This is the velocity of water over the rectangular crest on the assumption, generally true, that over the crest the flow is critical, with a depth \((z_g - f_g) = \frac{3Q^2}{gV_{cr}^2}\) (where \(Q\) is the design...
The water discharge in cumecs, \( l_c \) is the crest width in metres, and \( g \) is the acceleration due to gravity, in \( \text{m sec}^{-2} \). With reference to the symbols of fig. 31, the distance \( X \) of the free fall from the crest is:

\[
X = \sqrt{\frac{g(z_0 - f_0)}{2}} \frac{1}{g} \sqrt{2(z_0 - f_0)} \approx \sqrt{\frac{(z_0 - f_0)(z_0 - f_3)}{2}}
\]

In fig. 31 a weir without a counterweir is shown; in this case, \( z_3 = z_2 \) and \( f_i = f_3 \).

To simplify calculation, \( X \) has been plotted against \( (z_0 - f_3) \) in fig. 32.

With reference again to fig. 31, it is possible to evaluate the depth of scour \((z_2 - f_3) \), by means of an empirical formula, such as the most widely known one proposed by Schoklitsch [25], [26]:

\[
z_3 - f_b = 4.75 \frac{(z_0 - z_3)^{0.2} d_0^{0.57}}{d_i^{0.32}}
\]

where levels \( z_0, z_3, f_b \) are measured in \( \text{m} \) (where there is a sufficiently smooth downstream river bed, can be comparable to the free water surface in uniform flow), \( q \) is the unit discharge in cumecs per \( \text{m} \) width and \( d_i \) is the aperture diameter, in mm, of the sieve which passes 90% in weight, of the bed material.

In fig. 33 some curves with constant \( q \) and \( d_i \) are shown to enable the ready calculation of the depth of scour in relation to the fall of water. Through the examination of such curves, it can be seen that the depth of scour \((f_3 - f_0)\) can be easily reduced if the tailwater \((z_0 - f_3)\) is increased. This is achieved by constructing a counter-weir downstream at a distance from the weir and with a height \((f_i - f_3)\) sufficient to form a subcritical flow of depth \((z_2 - f_3)\); refer again to fig. 17.

In this case, the flow over the counterweir is given by the equation:

\[
Q \approx \mu l_i(z_2 - f_i)\sqrt{2g(z_2 - f_i)}
\]

where, as usual:

- \( Q \) is the rate of discharge, in cumecs, assumed for the design,
- \( \mu \) is the coefficient of discharge, ranging from 0.4 to 0.6, \( l_i \) is the width of the counterweir, in \( \text{m} \), \( f_i \) and \( z_2 \) are elevations, in \( \text{m} \). By fixing the elevation \( z_2 \) which keeps the depth of scour, and consequently the depth of the foundation within acceptable limits, equation (6) gives the elevation \( f_i \) of the counterweir.

The energy dissipation downstream of the counterweir, which can be assumed as the difference between the total heads in section 0 and 2, should be negligible compared with the dissipation in the pool, which in turn can be assumed as the difference between the total heads in section 0 and 2. If not, severe erosion could take place downstream of the counterweir, which if undermined would endanger the stability of the main weir itself.

As already stated, to evaluate the dimensions of the stilling pool it is necessary to know elevation \( z_3 \). Generally, it is calculated assuming uniform flow conditions, and in practice the actual characteristics of flow will approximate to these conditions, provided that there is no backwater effect caused by changes in the river cross-section in the vicinity of the weir. \( z_3 \) is calculated using the equation:

\[
Q = \chi \Omega \sqrt{Ri}
\]

where:

- \( Q \) (cumecs): design discharge;
- \( \Omega \) (\( \text{m}^2 \)): area of cross section of flow, for a depth of water \((z_3 - f_3)\);
- \( R \) (\( \text{m} \)): hydraulic radius, related to above depth;
- \( i \): slope of the river;
- \( \chi \) (\( \text{m}^{1/2} \text{sec}^{-1} \)): coefficient of resistance, related to above depth. It can be calculated using any of the empirical formulae given by Strickler, Basin, Kutter, or better Colebrook-White [15], [20], [27], [28], [29], [30], [31], [32]. Whichever equation is used, it is essential to determine the roughness of the river cross-section; reference should be made to the table of coefficients of roughness \( n \), included in Ven Te Chow's book [20].

The evaluation of \( z_3 \) using equation (7) is made by trial and error, adopting different values of \( z_3 \) until the design discharge is obtained.
3.2 Stilling pool with lined floor: 
jump control by broad crested weir

When the river bed consists mainly of loose material of limited size, or when the weir is such that a high degree of security is called for, it will be necessary to line the stilling pool, to prevent the bed from being scoured out.

A type of lined stilling pool is shown in fig. 18. The river bed is protected by a gabion apron, at an elevation $f_5$ almost coincident with the level $f_3$ of the river bed; $(f_5 - f_3)$ is the height of the counterweir above the apron.

Figs. 34, 35 - YUGOSLAVIA - S. Giorgio torrent (Istria) - Vertical gabion weir with lined stilling pool and counterweir.

The correct design of the pool requires that critical flow occurs on the counterweir so that the flow of water in the pool is not influenced by the flow downstream of the counterweir. Part of the energy of the water is dissipated immediately downstream of the counterweir and therefore in order to avoid scour of the river bed it is necessary to extend the apron downstream of the counterweir or to key it so deeply into the bed, that its stability is assured even if severe erosion takes place (figs. 34, 35).

Fig. 36 - Graph of $(z_1 - f_5)$ and $(z_2 - f_3)$. 
The dimensions of the pool are easily calculated using the following methods, since the water flows supercritically, with a depth \((z_1 - f_h)\), in section at the toe of the weir.

\((z_1 - f_h)\) is obtained from the equation of the hydraulic jump:

\[
(z_0 - f_h) + \frac{Q^2}{2g\Omega_b^2} = (z_1 - f_h) + \frac{Q^2}{2g(z_1 - f_h)^2 l_b^3}
\]

since there is no change in the total head between sections 0 and 1. As usual, elevations \(z_0\), \(f_h\) and \(z_1\) are in metres, \(Q\) is the discharge, in cumecs, \(\Omega_b\) is the area of cross section of flow, in \(m^2\), \(l_b\) is the width of the pool, in m.

In the above equation of the third degree, \((z_1 - f_h)\) is the unknown value. Generally, both the velocity head \(\frac{Q^2}{2g\Omega_b^2}\) and the depth \((z_1 - f_h)\) are small, if compared with the other terms, and can be neglected; depth \((z_1 - f_h)\) is therefore calculated from:

\[
(z_1 - f_h) \approx \frac{Q}{l_b \sqrt{2g(z_0 - f_h)}}
\]
The dissipation of energy occurs in the hydraulic jump, which must take place in the protected area between the weir and the counterweir. If $(z_1 - f_b)$ is the relative initial depth of the jump, the relative sequent depth is:

\[
(z_2 - f_b) = -\frac{(z_1 - f_b)}{2} + \sqrt{\frac{2Q^2}{g_0^2(z_1 - f_b)^2}} + \frac{(z_1 - f_b)^2}{4}
\]

This depth is obtained by means of a broadcrested weir (counterweir) at the end of the stilling pool.

The dimensions of the counterweir can be obtained from equation (6):

\[
Q = \mu (z_2 - f_c) L \sqrt{2g (z_2 - f_c)}
\]

where the unknown quantities are the elevation of the counterweir $f_c$ and its width $L$. Generally, $L$ does not differ much from $l_b$ and $l_g$ and therefore $f_c$ can be computed from equation (6), and finally the height $(f_c - f_b)$ of the counterweir can be found.
To simplify calculation, \((z_0 - f_b)\) has been plotted against \((z_0 - f_b)\) and \(Q/l_b\) and \((z_2 - f_b)\) has been plotted against \((z_1 - f_b)\) and \(Q/l_b\) in fig. 36.

The characteristics of flow downstream of the weir having been established it is necessary to verify that the tailwater does not affect the discharge over the crest. Of course, if submergence caused by the tailwater occurs, the jump is located further upstream, which is a safe condition. But in such case, it would be better to reduce the size of the pool, and to neglect the counter weir.

Another value to be found is the elevation \(z_e\) of the water adjacent to the downstream face of the weir. It can be computed approximately from the formula [26]:

\[
(z_e - f_b) = (f_u - f_b) \left[ \frac{Q^2}{g h^2 (f_u - f_b)^3} \right]^{0.22}
\]

Length of stilling pool. This is found by adding \(L_{g1}\), the distance of the weir from the position where the supercritical flow of depth \((z_1 - f_b)\) is formed, and \(L_{12}\) the length of the portion of pool where the hydraulic jump occurs.

For the computation of \(L_{g1}\), the points \(G\) of the axis of the nappe over the crest, \(V\) of the axis of the nappe cutting into the water in the pool, and \(P\) on the bottom of the pool at section 1, are assumed to be on a straight line (fig. 37).

It is also assumed that the nappe at \(G\) is horizontal and its velocity is critical (\(*\)); the loss of energy is not considered.

Hence, the projection of \(GV\) on the horizontal plane is:

\[
V'V = \sqrt{g(z_e - f_b) \left[ \frac{f_u + z_g - z_v}{2} \right]} - g =
\]

\[
= \sqrt{(z_e - f_b) (f_u + z_g - 2z_v)}
\]

For the value of \(z_g\) see paragraph 2; \(z_e\) is obtained from equation (11). Consequently, \(L_{g1}\) is:

\[
L_{g1} = GP \frac{V'V}{GV} = \left[ \frac{z_g + f_u - f_b}{2} \right] \sqrt{(z_e - f_b) (f_u + z_g - 2z_v)} \frac{z_g + f_u - z_v}{2}
\]

(12)

Obviously, \(L_{g1}\) is greater than the distance \(X\) calculated in equation (4). The length of that portion of basin in which the jump occurs:

(13) \(L_{12} = 6.9 [z_2 - f_b] - (z_1 - f_b) = 6.9 [z_2 - z_1]\)

The total length of the stilling pool is therefore:

(14) \(L_{yb} = L_{g1} + L_{12} =
\]

\[
= (z_g + f_u - 2f_b) \sqrt{\frac{z_g - f_u}{z_g + f_u - 2z_v} + 6.9 (z_2 - z_1)}
\]

An interesting study can be made of a weir backed by deposited material, up to the level of its (rectangular) crest (fig. 38).

In such a situation, the characteristics of flow can be expressed by simple equation as functions of the drop number \(D\) [18].

Such equations were developed by experimental investigations [33], [34], [35].

(15) \(D = q^2/g (f_u - f_b)^3\)

where \(q\) is the unit discharge flowing over the crest \((q = Q/l_b)\), \(g\) is the acceleration due to gravity, and \(f_u\) and \(f_b\) are shown in fig. 38.

The dimensions of the stilling pool can be derived from the following equations:

(16) \(L_{g1}/(f_u - f_b) = 4.30 \times 10^{-0.27}\)

(17) \((z_e - f_b)/(f_u - f_b) = 1.00 \times 10^{-0.22}\)

(18) \((z_1 - f_b)/(f_u - f_b) = 0.54 \times 10^{-0.425}\)

(19) \((z_2 - f_b)/(f_u - f_b) = 1.66 \times 10^{-0.27}\)

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

To simplify calculation, the drop number \(D\) has been plotted against \((f_u - f_b)\) and \(q\) in fig. 39, and the values necessary for dimensioning the stilling pool may be obtained from the curves in fig. 39.

(*) Actually this condition occurs not at the edge of the crest, but upstream of it at a distance of 3-4 times \((z_e - f_b)\). However, such an approximation is sufficient at least for small structures. For important weirs, it is advisable to verify the hydraulic behaviour of the structure using a model.
3.3 Stilling pool with lined floor: jump control by abrupt rise

The control of the jump can be achieved by means of an abrupt rise. In this case, the tailwater influences the conditions of flow in the pool. Following the layout shown in fig. 19, the bottom level of the pool is below \( f_b \) the elevation of the natural bed.

The flow in the pool may be described mathematically in terms of the following equations:

\[
Q = \mu l_b (z_0 - f_b) \sqrt{2g(z_0 - f_b)} \quad \text{over the crest;}
\]

\[
(z_0 - f_b) + \frac{Q^2}{2g\Omega_0^2} = \frac{Q^2}{2g(z_1 - f_b)^2 l_b^2} \quad \text{between section 0 and section 1;}
\]

\[
(z_2 - f_b) = -\frac{(z_1 - f_b)}{2} + \sqrt{\frac{2Q^2}{gl_b^2(z_1 - f_b)} + \frac{(z_1 - f_b)^2}{4}} \quad \text{(21)}
\]

\[
Q_3 = \frac{Q_2}{Q_2} \quad \text{(hydraulic jump)}
\]

\[
(z_3 - f_b) + \frac{Q^2}{2g\Omega_3^2} \geq \frac{Q^2}{2g(z_2 - f_b)^2 l_b^2} \quad \text{between section 2 and section 3}
\]

For the meaning of the symbols, reference should be made to the equations developed in the preceding paragraphs, and to fig. 19; \( \Omega_3 \) is the area of cross section of flow downstream of the pool. Equation (21) is not strictly true, because the total head must decrease as one proceeds downstream; however, by imposing the condition that the total head in section 2 is smaller than the total head in the river downstream, it is possible to determine the correct height for the rise necessary to prevent the jump from being drowned.

3.4 Construction details

The following recommendations should be followed wherever possible:

- The apron of the stilling pool should be constructed of two layers of gabions, each being 0.50 or 0.30 m high. This double layer will give a better performance, and will allow speedy and economical maintenance should unusually severe floods, carrying heavy bed load, damage the upper layer.

- The gabions in the apron of the stilling pool should be filled with large stones (20-30 cm) preferably rounded. Careful attention should be given to the filling operation to ensure the minimum of voids. Together, these simple precautions will prevent the force of the water from producing unacceptable settlement, or movement of the fill inside the compartments (figs. 18, 19).

- The side slopes adjacent to the weir should be protected from scour, especially if the river is narrow and its banks are easily eroded. Either sloping revetments or side walls may be adopted and the protection can extend upstream and downstream if thought necessary. Such protection should not be connected with the downstream apron, as this must be left free to deflect downward.

(a) Equation (3) was discussed in para. 2.
(b) In equation (8) and (10), the discharge $Q$ and the characteristics of flow in section 0 are known, $z_1$ and $z_2$ are the unknown quantities, while the elevation $f_b$ is obtained by trial and error.
(c) In equation (21) $h_b$ is given, $z_3$ and $\Omega_3$ are determined by the conditions of flow downstream and as previously stated equation (21) allows one to verify the chosen value of $f_b$.

If the weir causes a deposition of material upstream up to crest level $f_a$, the calculations for the stilling pool are as developed in para. 3.2 [equations (15) to (20)]. As usual, the conditions of flow in the pool will be influenced by the tailwater level (figs. 40, 41).
Gabion weirs may be stepped (figs. 42, 43). When compared with the vertical type, the advantages of a stepped weir are: (1) better stability, due to the more rational cross-section, and (2) the dissipation of some energy on each step, which may be of advantage when considering the stilling pool design. The pool itself can be shortened, or even neglected, if the height and length of the steps are such as to allow complete energy dissipation by means of an hydraulic jump at each fall, thus forming a ladder of cascades [20]. This situation may be obtained by using pooled steps with counterweirs, or inclined steps (figs. 44, 45).

Hydraulic calculations of such dissipators have been extensively studied by B. Poggi (pooled steps) [36], [37] and by CIRIA (inclined steps) [38]. Design criteria developed in the last mentioned report may be applied to the structures under consideration (having an average slope of the downstream stepped face of between 1:1 and 2:1) only when the unit discharge is small and there are a large number of steps. Due to technical and economical reasons, the stepped weir seldom has characteristics similar to those described above and therefore it is generally difficult to determine the energy dissipated at each step and consequently the residual energy at the toe of the spillway.

Research on the evaluation of the residual energy at the base of a stepped spillway, having a limited number of steps and in the presence of high discharges, was carried out by Dr. D. Stephenson, who particularly developed his study on the use of stepped gabion weirs as energy dissipators at the outlet of dams [39], [40], [41], [42], [43].

These studies were carried out using a model; a procedure always advisable when determining the actual behaviour of stepped weirs, and their associated stilling pool, of some importance. Whenever model tests are not available, it is wise to neglect the dissipation of energy on the steps and to design the stilling pool as for a vertical weir. Not only are stepped weirs subject to uncertain design procedures, they are subject also to possible damage to the steps. Solid materials carried by the stream may eventually abrade and fracture the gabion mesh. Moreover the water itself hits the horizontal surfaces...
of the steps and can displace the fill, causing bulges or voids.

For such reasons, it is advisable to employ stepped weirs only in rivers having small discharges per unit width and carrying little solid material. They are not suitable in rivers carrying heavy boulders in spate conditions, where vertical or sloped weirs are recommended, as these are better hydraulically and have a longer life.

![Diagram of stepped weir with inclined steps](image1)

1: River bed  
2: Water surface  
3: Stepped weir with inclined steps

![Diagram of stepped weir with pooled steps](image2)

1: River bed  
2: Water surface  
3: Stepped weir with pooled steps  
4: Sill

Figs. 44, 45 - Correct design of stepped gabion weirs.

### 5 The control of seepage and the prevention of undermining

As mentioned in the foreword, gabion weirs are used particularly where loose or fine grained soils, having high permeability are found. Since a weir causes the upstream head to rise, water tends to seep under and around the structure.

The problem is to minimize and control this seepage, since, if this flow has a velocity capable of removing individual particles of the foundation soil, harmful effects take place. Water reaching the discharge surface forms soft spots and leaches out the fine particles and if no preventative measures are taken, the structure may be undermined or outflanked. In order that a proper study of the seepage may be made, the flow net through the foundation soil must be drawn. Where a gabion structure is not isolated from the soil by an impermeable layer, the flow net through the structure must be included in the study.

The construction of the net (i.e. of flow lines and equipotential lines) allows both the velocity of seepage and the hydraulic head at any point to be ascertained.
The velocity of seepage must be consistent with the equilibrium of the smallest particles of the foundation soil.

The flow net can be constructed using the various equations governing the flow of water through soil, or by means of an electric analogy test which takes advantage from the analogy between Ohm's and Darcy-Ritter's equations. Details of this problem are widely described in specialized text books [44], [45].

For the preliminary design of small weirs, quicker empirical methods for studying seepage can be employed. A well-known method is the one given by "Bligh's equation": according to this, the total path \( L \) of seeping flow under and around the structure must be:

\[
L > c \Delta h
\]

where:

\( \Delta h \): difference between the upstream and downstream water surfaces.

\( c \): a coefficient depending on the type of soil. Obviously the worst soils are silts and muds which are permeable and due to their small particle size, easily washed out. The best soils are impermeable compacted clays, and gravels and boulders which, although permeable, are only moved by very high flow velocities. The recommended values of \( c \), relating to different types of soil, are set out in table 3.

Table No. 3 - Values of coefficient \( c \) for control of seepage [46].

<table>
<thead>
<tr>
<th>( c )</th>
<th>Size of particles (mm)</th>
<th>Type of soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.01-0.05</td>
<td>Fine silt and mud</td>
</tr>
<tr>
<td>18</td>
<td>0.06-0.10</td>
<td>Coarse silt and very fine sand</td>
</tr>
<tr>
<td>15</td>
<td>0.12-0.25</td>
<td>Fine sand</td>
</tr>
<tr>
<td>12</td>
<td>0.30-0.50</td>
<td>Medium sand</td>
</tr>
<tr>
<td>10</td>
<td>0.60-1.00</td>
<td>Coarse sand</td>
</tr>
<tr>
<td>9-4</td>
<td>2.00</td>
<td>Gravel</td>
</tr>
<tr>
<td>6-3</td>
<td>0.005</td>
<td>Hard clay</td>
</tr>
</tbody>
</table>

Fig. 46 - Flow net for a gabion weir.

Fig. 47 - ITALY - Castel S. Pietro (Bologna) - Laying of filter cloth under the foundation and wings of a weir.
As the permeability of gabions is higher than that of the surrounding soil, gabion structures behave as drains and collect the water seeping through the foundation soil and through the soil trapped by the structure.

Such a situation is shown in fig. 46: the flow lines, appearing at the interface between soil and gabions at an elevation higher than $Z_3$, are at atmosphere pressure. Those reaching the surface at an elevation lower than $Z_3$, however, are subject to a head equal to $Z_3$, as the head loss of water seeping through the gabions is negligible when compared to the loss through the soil. In such conditions, due to the steep hydraulic gradient and high filtration velocity, the finest particles will be transported into and through the gabions themselves. This leaching of material could cause the collapse of the structure and must therefore be prevented.

The surest way to avoid undermining is to construct an impermeable cut-off, under, and at the sides of, the structure, deep enough to reach the impermeable layers of soil (see chap. IV para. 3.2.1a “Weir on the Brenta River”).

When technical or economical reasons make the construction of a cut-off impossible or inconvenient, other methods may be used, as explained below.

All such methods aim to reduce the velocity of the seepage flow and/or control the leaching of the fine particles. The velocity of seepage may be reduced by lengthening the flow path thus reducing its gradient and, consequently, its velocity. An impermeable membrane placed under and behind the gabion work can be used for this purpose.

The control of leaching of fines through the gabion work is obtained by placing filters under the structure, thus allowing water, but not the bed material, to pass through.

Generally, various layers of coarse sand and gravel are used. Recently, however, synthetic filter clothes (usually “non-woven filter clothes”) have been employed on a large scale. Such filters are available in different thickness, depending on the specific need; they are non-rotting and are not attacked by insects or rats. Laying is easier and quicker than stone filters and therefore they are generally more economical (fig. 47).

Sand asphalt, a porous mix of sand with a small quantity of bitumen and filler, may also be used as a filter. Its use is particularly suitable where the filter is to be placed underwater, or when the banks are to be regraded. All filters, of whatever material, tend to become obstructed in the long run by the continuous concentration of fine materials. It is therefore advisable to apply Bligh’s formula even with permeable filters, considering them as impermeable layers.

Where a ramp of earth or clay is constructed against the upstream face, the membrane, or filter, is put between the ramp and the structure in order to avoid the washing away of

Fig. 48 - Control of seepage according to Bligh’s formula, for a weir built on permeable soil.
the ramp. A ramp is recommended whenever the crest level is above the upstream bed, to increase the stability of the structure and protect it from the impact of water and where the construction of a cut-off is not convenient (*).

In all cases the control of seepage under the structure is based on the assumption that the interface between gabions and foundation soil is impermeable over its whole length. If the conditions indicated by Bligh's formula are not satisfied, it is necessary to increase the length of the flow lines by adding aprons or cut-offs.

Since the length of any integral stilling pool is designed to ensure that the hydraulic jump occurs within the pool, it should not be increased in order to increase the length of the flow lines. Upstream and downstream aprons (with filters) should be provided to achieve this (fig. 48).

(* This upstream ramp is necessary both in weirs for river training, where it protects the structure from the dynamic action of water, and in weirs built for water diversion or storage as it increases the storage capacity of the structure. Obviously, it should not be used in cases where it is necessary to maintain the high permeability of gabions, e.g., in the construction of gabion energy dissipators at the outlet to dams [39], [40], [41].

6 Structural stability

A gabion weir is considered to be a mass gravity structure, bearing on the foundation soil and subject to a series of horizontal forces (upstream and downstream water and soil pressure) and vertical forces (weight of the structure, weight of soil on the steps, weight of water on the crest and on the steps, and uplift pressure).

Exceptionally other forces, like those exerted by earthquakes, landslides or frost, may act on the structure.

The design considerations for the stability of a gabion weir are generally the same as for any mass gravity rigid structure in, say, reinforced concrete or masonry. The fundamental characteristics of the gabion i.e. its flexibility and its ability to re-distribute forces and pressures, are not taken into account [47].

A check on stability must be made first of all at the section under the crest, where the height above foundation level is usually greatest; a similar check is then made for a section of the wings, where conditions are usually less severe.

All the factors affecting the stability are considered below.

a) Unit weights

— *Water*: normally, the density of water $\gamma_w$ varies between 1000 and 1100 kg/m$^3$, but it can reach in excess of 2000 kg/m$^3$, depending on its turbidity. This must be taken into account especially when considering structures used in the training of torrents.

— *Gabions*: boxes made of hexagonal steel wire mesh, filled
with river boulders or quarry stones. The mass of the mesh can be neglected since it is minute when compared with the filling material. Any type of hard and durable stone can be used for filling the gabion, and table 4 gives the indicative density \( \gamma_s \) of some of the most common filling materials.

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Unit weights (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt</td>
<td>2900</td>
</tr>
<tr>
<td>Granite</td>
<td>2600</td>
</tr>
<tr>
<td>Hard limestone</td>
<td>2600</td>
</tr>
<tr>
<td>Trachytes</td>
<td>2500</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2300</td>
</tr>
<tr>
<td>Soft limestone</td>
<td>2200</td>
</tr>
<tr>
<td>Tuff</td>
<td>1700</td>
</tr>
</tbody>
</table>

When the density of the filling material and the porosity \( n(*) \) are known, the unit mass of the gabion structure is:

\[
\gamma_g = \gamma_s(1 - n)
\]

The porosity \( n \) is about 0.3 in most cases. For easy calculation, the density of gabions \( \gamma_g \), related to the density of stone fill \( \gamma_s \), may be found from the curves in fig. 49, for different values of \( n \).

If the voids are partially filled with water, and \( u(**) \) is the degree of saturation the density is:

\[
\gamma_{gw} = \gamma_s(1 - n) + n\gamma_w
\]

which becomes \( \gamma_{gs1} = \gamma_s(1 - n) + n\gamma_{w1} \), the density of gabions saturated with water (**), i.e. when \( u = 1 \).

Example: gabions, filled with stone of \( \gamma_s = 2500 \text{ kg/m}^3 \), with a void content of 30% will have \( \gamma_g = 1750 \text{ kg/m}^3 \) and a \( \gamma_{gs1} = 2050 \text{ kg/m}^3 \).

- **Soil**: like gabions, soil has a density depending on the specific gravity of its individual grains, on porosity \( n \) and on the degree of saturation \( u \):

\[
\gamma_{is} = \gamma_s(1 - n) + n\gamma_w
\]

which becomes \( \gamma_{is0} = \gamma_s(1 - n) \), the density of dry soil, when \( u = 0 \), and \( \gamma_{is1} = \gamma_s(1 - n) + n\gamma_{w1} \), the density of saturated soil, when \( u = 1 \). For submerged soil, the unit weight is: \( \gamma_{iw} = \gamma_s(1 - n) \).

b) **Horizontal thrust**

- **Hydrostatic pressure**. The following is based on the assumption of a correctly designed, vertical weir i.e., having the upstream ramp separated from the gabion work by an impermeable membrane which is continued under and on each side of the weir (fig. 50). In fact it is correct to follow this calculation procedure even where the weir is not separated from soil by any membrane or filter as, in the long run, the soil adjacent to the structure tends to consolidate, and effectively separates the gabions and the surrounding soil.

A section of structure 1 m wide, limited by \( E - E' \), \( F - F' \), and \( E - F \), is considered.

Besides the hydrostatic pressure:

\[
H_{w2} = \frac{1}{2} \gamma_w h_2^2
\]

acting on that part of the structure above soil level, pressure

\[ (*) \] Porosity \( n \) is the ratio between the volume of voids and the total volume of the structure, or of the soil.

\[ (**) \] Degree of saturation \( u \) is the percentage of the volume of voids filled with water.

\[ (***) \] Where the gabions are under water, their weight is diminished by the uplift force acting on the stone filling. The density of submerged gabions \( \gamma_{gw} \) is therefore:

\[
\gamma_{gw} = (\gamma_s - \gamma_w)(1 - n)
\]
$H_{W1}$ and $H_{W3}$ act on that part of structure below soil level due to the presence of the seepage water (*).

The worst condition occurs when the gabion work is separated from the soil by an impermeable membrane, or by a clogged filter. In such a case, water seeps through the soil under and along the sides of the structure, and the hydraulic head varies from a maximum $z_M$ upstream, to a minimum $z_V$ downstream. By drawing the flow net the pressure can be estimated at each point and consequently the uplift pressure may be determined. If the flow net is not available, a quicker, and generally more conservative calculation, is made by supposing the distribution of uplift pressure on the upstream and downstream face, as indicated by the dashed lines in fig. 50; this is based on the assumption that head losses are concentrated along the foundation.

With these assumptions, the respective resultants of hydrostatic pressure $H_{wM}$ on the upstream face, $H_{wV}$ on the downstream face, are:

\begin{align}
(27) \quad H_{wM} &= H_{w1} = \frac{1}{2} \gamma_w [(h_1 + h_2 + h_3)^2 - h_1^2] \\
(28) \quad H_{wV} &= H_{w2} + H_{w3} = \frac{1}{2} \gamma_w [(h_4 + h_3)^2]
\end{align}

These pressures could be calculated more accurately by supposing a linear variation along the foundation between the upstream value $z_M$ and the downstream value $z_V$.

— Soil pressure: the horizontal pressure of upstream soil is calculated neglecting the friction between soil and gabions, and considering the “active” state. Referring again to fig. 50(**):

\begin{align}
(29) \quad H_{tM} &= \frac{1}{2} \gamma_{lt} (h_2 + h_3)^2 \lambda_a \\
&= \frac{1}{2} \gamma_{lt} (h_2 + h_3)^2 \tan^2 \left( \frac{\pi}{4} - \frac{\phi}{2} \right)
\end{align}

where $\lambda_a = \tan^2 \left( \frac{\pi}{4} - \frac{\phi}{2} \right)$, and $\phi$ is the angle of internal friction of soil. This force is applied at $\frac{h_2 + h_3}{3}$ above foundation level.

The downstream soil pressure is also calculated considering the «active state», or may be neglected:

\begin{align}
(30) \quad H_{tV} &= \frac{1}{2} \gamma_{lt} h_3^2 \lambda_a
\end{align}

$H_{tV}$ is applied at $h_3/3$ above foundation.

c) Hydraulic uplift and forces exerted by soil and water on the steps.

It is easy to determine the intensity of vertical forces acting on upstream or downstream steps (for instance, forces $P_{w2}$ and $P_1$ in fig. 50), and of the uplift pressure under the foundation ($S_w$ in fig. 50).

If a hydrostatic distribution of pressure is considered as acting on the upstream and downstream faces of the structure, the uplift pressure $S_w$ is the resultant of the trapezoidal pressure diagram: $\gamma_w (h_1 + h_2 + h_3)$ upstream and $\gamma_w (h_4 + h_3)$ downstream.

\begin{align}
(*) \quad \text{Pressures due to submerged soil and to water are considered separately. For example the pressure on the soil at an elevation } z \text{ is given by (fig. 50):}
\end{align}

\begin{align}
P_{soil} &= \gamma_w (z - f_u) + \gamma_{lt} (f_u - z) = \gamma_w (z - u) + \gamma_{lt} (f_u - z),
\end{align}

where $\gamma_w (z - u)$ is the hydrostatic pressure and $\gamma_{lt} (f_u - z)$ is the soil pressure.

\begin{align}
(**) \quad \text{Same procedure applies when considering a gabion weir without an upstream ramp, but partially or totally backed with naturally deposited bed material.}
\end{align}
6.1 Stability against overturning

In the following notes, the structure sketched in fig. 50, with upstream ramp and separated from the soil by an impermeable or a synthetic or reverse filter, will be considered as typical. The same criteria can be used for gabion structures of different design, having due regard for the variation in hydraulic behaviour. For example, a gabion weir with a foundation protected against seepage or on an otherwise stable foundation but without any upstream ramp is at first permeable and allows a large part of the flow to pass through it. Later, should the structure trap sufficient suspended material to seal itself, the water will flow only over the crest, while seepage water will tend to be drained by the gabion structure. It is therefore obvious that the worst situation occurs immediately after construction, and for a proper statical and hydraulic calculation, it will be necessary to evaluate the percentage of the discharge flowing through the gabion structure above soil and that flowing over the crest, and to determine the seepage line of water flowing through the structure. For useful information on this subject, refer to [39], [40], [41]. Returning then to the typical structure (fig. 50) the design analysis is as follows.

The structure tends to overturn around point F and therefore stability against overturning is ensured if the resultant of the forces cuts the foundation to the left of point F, i.e. when the anti-clockwise moment of stabilizing forces is greater than the clockwise moment of the overturning forces.

The stabilizing forces are:
- dead weight of the structure \( P_d \). Since the structure is sealed by an upstream ramp and as gabions are much more permeable than soil, all that part of the weir above tailwater level can be considered dry (density: \( \gamma_d \)), while that part of the weir below it is submerged (density: \( \gamma_s \)).
- weight of water on the crest \( P_{w1} \).
- weight of water \( P_{w2} \) and of soil \( P_s \) (of density \( \gamma_s \)), on the upstream and downstream steps.
- horizontal thrust \( H_w \) of water and \( H_s \) of soil, on downstream faces as calculated at paragraph 6 (b).

Overturning forces are:
- horizontal thrust \( H_{wM} \) of water and \( H_{sM} \) of soil on upstream faces as previously calculated.
- hydraulic uplift \( S_w \), as considered in para. 6 (c);
- other forces which might occasionally act on the weir, such as: frost heave, dynamic action of supercritical flows, seismic forces, sliding soil pressure and also the possible effects of downstream scouring caused by the falling nappe.

If \( M_o \) is the moment of overturning forces and \( M_r \) the moment of restoring forces about \( F \), the coefficient of stability against overturning is the ratio:

\[
s_r = M_r / M_o
\]

For stability, it is necessary that \( s_r \geq 1 \), i.e. \( M_o \geq M_r \). In the case of small weirs, it is sufficient that \( s_r \geq 1.3 \); higher coefficients of stability must be considered in the case of more important structures.
6.2 Stability against sliding

Reference is made again to fig. 50. The structure will fail in sliding, when the sum of the horizontal forces acting at the base is not balanced by the friction and cohesion forces between the base and the foundation soil. The horizontal forces are water pressures \( H_wM \) and \( H_wV \) and soil pressures \( H_{IM} \) and \( H_{IV} \), as calculated previously, plus any other infrequent forces (ice, etc.).

The stabilizing force is mainly due to friction, since cohesion between gabions and foundation soil is virtually nil. This force is given by the resultant of the vertical forces multiplied by the coefficient of friction between the structure and the soil. These vertical forces are: the dead weight of the structure \( P_g \), the weight of water on the crest \( P_{wl} \), the weight of water \( P_{wz} \) and of soil \( P_t \), on the upstream and downstream steps of the structure, the hydraulic uplift \( S_u \), plus any other forces (for instance: forces due to seismic actions). Stability against sliding is ensured when:

\[
\sum H < \sum V \times \tan \phi
\]

As shown in section (5) lining the stilling pool is usually necessary to protect against seepage failures.

Where this lining is constructed with mastic grouted Reno mattresses, or gabions or mattresses laying on a reverse or synthetic filter, it is necessary to check the stability of the lining against uplift, i.e. to check that the uplift force due to seepage water is not greater than the combined weight of the lining and of the water passing over it.

It is therefore necessary to evaluate the distribution of pressures under the stilling pool by drawing the flow-net diagram or by using the simplified method already suggested (assuming a linear variation in head along an impervious foundation). With reference to fig. 48, the pressure \( P \) at each point of the foundation is

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

If \( h \) is the depth of water above the apron, \( s \) the thickness of the apron, the coefficient of stability against uplift \( s_g \) is:

\[
s_g = \frac{(\gamma_{so} s + \gamma_w h)}{p};
\]

Acceptable values of \( s_g \) are 1.1-1.2.

\*\* It is of interest to note the results of tests performed by the Instituto Costarricense de Electricidad [48], during the study of the Rio Reventado training project. Gabions were placed on a layer of clay saturated by water, and the force necessary to cause horizontal movement was ascertained. A coefficient of friction of approx. 0.75 was indicated.

6.3 Stability against uplift
The distribution of pressures is calculated for a section under the crest, which is usually the position of highest stress. Both the distribution of pressures on the foundation soil and the related pressure on the gabions at each layer and their resistance are studied.

7.1 Pressure on foundation soil

For each section under examination all the forces $H_{w',v}, H_{v,v}, P_{w',v}, P_{v,v}, P_{w,v}, P_{w}$, $S_w$ of fig. 50 are computed for the worst case. The resultant $R$, its inclination and the centre of pressure $X$ are then found. It is conservatively assumed that the gabion foundation surface remains linear (i.e. the pressure diagram is linear) and that the foundation soil is much less rigid than the gabion structure. With regard to this second assumption, the results of recent tests in progress [47] show that the rigidity of gabions is comparable to that of soil.

If the centre of pressure $X$ is within the middle third $MN$ (fig. 51), the pressure is distributed over the whole foundation, and the maximum pressure $\sigma_B$ at the downstream toe $B$ is, in kg/cm²:

$$\sigma_B = \frac{V \times XM}{100 \times AB^2}$$

(35)

where $V$ is the vertical component of the resultant $R$, in kg, and $XM$ and $AB$ distances, in cm.

If the centre of pressure $X$ is coincident with the extreme edge of the middle third $(N)$, the maximum pressure $\sigma_B$ is:

$$\sigma_B = \frac{2V}{100 \times AB}$$

(36)

A centre of pressure $X$ external to the middle third $MN$ (fig. 52) 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). However, in such a case, the pressure $\sigma_B$ would be:

$$\sigma_B = \frac{2V}{3 \times XB \times 100}$$

(37)

The maximum foundation pressure should be less than $K_i$, the bearing capacity of the soil, (see table 5).
Table No. 5 - Bearing capacity of soil [49].

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Bearing capacity $K_r$ (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Not compacted, borrow soils:</td>
<td>0-1.0</td>
</tr>
<tr>
<td>2) Loose compacted soils:</td>
<td></td>
</tr>
<tr>
<td>a) sand, grain diameter less than 1 mm</td>
<td>2.0</td>
</tr>
<tr>
<td>b) sand grain diameter 1 to 3 mm</td>
<td>3.0</td>
</tr>
<tr>
<td>c) sand and gravel (of at least 1/3 gravel)</td>
<td>4.0</td>
</tr>
<tr>
<td>3) Cohesive soils (classified according to the water content):</td>
<td></td>
</tr>
<tr>
<td>a) fluid; plastic fluid</td>
<td>0</td>
</tr>
<tr>
<td>b) plastic soft</td>
<td>0.4</td>
</tr>
<tr>
<td>c) plastic solid</td>
<td>0.8</td>
</tr>
<tr>
<td>d) semisolid</td>
<td>1.5</td>
</tr>
<tr>
<td>e) solid</td>
<td>3.0</td>
</tr>
<tr>
<td>4) Rock in good conditions (if fissured the indicated bearing capacity must be reduced to less than half):</td>
<td></td>
</tr>
<tr>
<td>a) sandstone, limestone, volcanic rocks, etc.</td>
<td>10.0-15.0</td>
</tr>
</tbody>
</table>

For important structures, it is appropriate to study not only the pressure on the soil but also to study the resistance of the soil to shearing stresses.

In fact, when the weir overloads the foundation soil, this tends to fail along surfaces having the approximate shape indicated in fig. 53.

Fig. 53 - Checking of shear resistance of foundation soil.

The maximum load which can be applied to the foundation $AB$ without causing failure by shear, depends on the resistance to shearing stress of the soil $T_f$:

$$T_f = c + \sigma \tan \phi$$  \hspace{5cm} (38)

If the maximum $T_f$ (Coulomb's) is reached, a surface of sliding is formed.

To each possible surface of sliding $BE$, a particular value of the resistance to shearing stress $Q$ is associated:

$$Q = \int_B^E \tau_f \, ds = \int_B^E (c + \sigma \tan \phi) \, ds$$  \hspace{5cm} (39)

There is a surface for which a minimum value of $Q$ is found: this is the approximate value of the resistance of soil to shear. The coefficient of safety $s_r$ against shear is:

$$s_r = \frac{Q}{V}$$  \hspace{5cm} (40)

where $Q_L$ is the vertical component of $Q$, and $V$ is the vertical component of the resultant $R$. For $s_r$, a minimum value of 2 is recommended. For more extensive information on the study of critical shear surfaces and shear resistance of soil, specialized texts should be examined [50], [51], [52].
7.2 Bearing resistance of the gabion structure

Checking the bearing pressure on the gabion structure is generally unnecessary in weirs of limited height, provided that the stability of the whole structure has been extensively checked and the pressure applied to the foundation soil, which usually has a resistance much less than that of the structure, is acceptable. If required, the bearing pressure on the gabions can be calculated in the same way as for that on foundation soil: in any horizontal section $AB$ at a section under the crest, the vertical component $V$ of the resultant $R$ of all forces acting on the section under study and its centre of pressure $X$ are again obtained. Equation (35), (36) and (37) give the maximum pressure on the gabions, where the centre of pressure $X$ is respectively within, at the boundary $N$, or outside the middle third, and making the safe assumption that the gabion structure does not withstand tensile stresses.

The maximum pressure on the gabions is then compared with the bearing capacity of the gabion structure itself. What that bearing capacity is can be specified only with the aid of tests, as it depends on a series of factors, whose influence is difficult to evaluate, i.e. shape and disposition of the stone fill, mechanical resistance of the fill, tensile strength of the steel wire mesh, etc.

The previously mentioned tests [47] recently carried out at the University of Bologna on full size gabion elements filled normally with river boulders, confirmed that the gabion has an excellent capacity for deformation and a high degree of resistance.

Compressive tests with deformation not allowed on two opposite sides, representing an uniaxial stress condition, indicated that a gabion is able to withstand differential settlements $\Delta h/h$ of around 20%, where $h$ is the initial height of the element with a load $P$, applied to the surface of initial area $A$, of about 10 kg/cm², and the modulus $d(\Delta P/A)/d(\Delta h/h)$ approximating to 50 kg/cm².

Such settlement took place without any mesh wire failing. In fact only when the applied load reached 30 to 40 kg/cm² did any of the mesh wires break.

8 Stability and resistance of the wings

As stated at para. 6, the worst conditions for stability and resistance generally occur at the central part of the weir. Moreover, when the wings are properly designed so that they cannot be over-topped, they may be stepped downstream and this is an advantage to the statical behaviour of the structure.

In any case, it is wise to check the true situation at the wings and a check on the stability and resistance should be carried out as described in the previous paragraphs 6 and 7.
CHAPTER III
Design criteria for sloped weirs.
Construction details

1 General criteria

Generally speaking, weirs with a sloped downstream profile are used for bed stabilization in river training schemes, and as impermeable structures to create storage reservoirs of between 5 and 15 metres deep. Since they have relatively large foundation areas, they are very suitable for heights of less than 5 m on poor soils where a fairly uniform pressure distribution is required. In this connection a weir having both upstream and downstream faces sloped, gives improved stability and hydraulic performance compared to a vertical structure (*)

The crests, downstream glacis and stilling ponds of these weirs are usually faced with an impermeable lining to prevent the migration of soil through the gabions, to protect the wire mesh against abrasion by debris and to consolidate the stone fill which could otherwise be displaced by water at high velocities particularly at the toe. The wing walls which must be high enough to ensure that they are not overtopped, should also be lined or backed by synthetic filter membranes. Concrete can be used for the lining, but a reasonable period, depending on the design of the structures and the foundation soil, should be allowed for any major settlement to take place before it is laid.

(*) The reason for the limiting height of 15 m is an economical and not a technical one. Weirs higher than this should be constructed with a central core of rock or other selected fill [53].
Bituminous sand mastic is preferable in view of its flexibility which allows it to move with any change in the gabion structure, and also because of its greater impermeability: (figs. 54, 55, 56).

Other examples of similar weirs are shown in chapter IV. The design criteria do not differ greatly from those for vertical faced weirs, and only the variations are discussed below.

2 Crest and stilling pool design

The hydraulic calculations for a sloped weir differ from those described in fig. 18 and 19 only because the water does not form a free overfall, but flows down the downstream face. The slope of the glacis is generally 1:2 or 2:3, which is sufficient to guarantee that the flow adheres to the surface even in conditions of high unit discharge.

For the computation of the dimensions of the crest and stilling pool, reference is made to paragraphs 3.1, 3.2, and particularly to equation (3), (6), (8), (10), (21).

The length of the stilling pool is 6.9 \( (z_2 - z_1) \); refer to fig. 57.

The subject of seepage is covered in paragraph 5.
3 Stability and pressure distribution

It is not necessary to check the stability against overturning; and no verification of pressure distribution on the gabion structure is necessary if that on the soil is acceptable. For high structures founded on poor soils, the distribution of pressure on the soil should be fairly uniform. Otherwise the only points to be considered are the safety factor against sliding (see para. 6.2); and the pressure on the foundation soil (para. 7.1), (both should be checked on sections at right angles through the crest and through the wing walls), and the stability of the stilling pool against uplift, (para 6.3).
CHAPTER IV
Examples of completed works

Examples of gabion weirs throughout the world are described in this chapter, and calculations are shown for the structures in sections 1.1.1a), 1.1.1b), 2.3g), 3.2.1a).

1 Vertical weirs

1.1 Stilling pool with unlined floor

1.1.1 Without counterweir

a) Italy, Castel S. Pietro (Bologna) - River training on the Rio del Coniglio torrent - 1970.

The Rio del Coniglio torrent is a tributary of the River Sillaro, located in the Apennines near Bologna where the soil mainly consists of very soft clay (fig. 58).

The upper reach of the torrent is deeply cut through eroded hills, which are subject to frequent slips caused by continuous scouring. At the point where the torrent reaches the plains and the confluence with River Sillaro, there had been massive deposition and raising of the river bed, which could have caused dangerous flooding.

The torrent was trained by erecting two check dams 5 m high (fig. 59, 60, 61).

In the following pages, a brief description of these structures is given, and the design calculation for one of them. In the reach where the weirs were erected, the torrent had a slope \( i \) around 3.5% and a width \( l \) about 20 m. Taking a discharge \( Q = 20 \) m\(^3\)/sec (corresponding to a unit discharge of 3.5 m\(^3\)/sec per sq km of catchment area) and a roughness coefficient in the river \( C = 30 \) m\(^{1/3}\) sec\(^{-1}\) (Strickler), from equation (7) the depth \( y \) of uniform flow is computed as:

\[
y = \left( \frac{Q}{Cl^{1/2}} \right)^{3/5} = \left( \frac{20}{30 \times 20 \times 0.035^{1/2}} \right)^{3/5} = 0.355 \text{ m}
\]

The mean velocity is:

\[
V = \frac{Q}{ly} = \frac{20}{20 \times 0.355} = 2.815 \text{ m/sec}
\]

Fig. 58 - Sketch map of the Rio del Coniglio torrent, tributary of the Sillaro torrent. Sketch map of the Savena torrent.
which is higher than the critical velocity

\[ \sqrt{gy} = \sqrt{9.81 \times 0.355} = 1.87 \text{ m/sec} \]

The mean velocity \( V \) is sufficient to move heavy material. The object of the training project was to obtain a bed slope of about 1 to 1.5\% and a mean velocity of approximately 2 m/sec, which apply to other torrents having similar characteristics, and which provide stable conditions for the bed materials.

The dimensions of the weir are shown in figs. 62, 63. The structure was backfilled with compacted soil.

Using the method set out in chapter II, the calculations are given below:

A) Crest design

A trapezoidal crest was chosen, the top being protected by concrete (figs. 28, 29).

The base was 8 m wide and the side slopes at 3:4.

The flow over the crest is given by the equation for critical flow, in which \( (z_a - f_a) \) is the depth of water, \( \Omega_a \) the area of the cross section of flow, \( b \) the width of water surface and \( Q_c \) the critical discharge

\[ Q_c = \Omega_a \sqrt{g \Omega_a / b} \]

for which following values are obtained using the above figures.

<table>
<thead>
<tr>
<th>( Q_c (m^3/sec) )</th>
<th>2.28 6.56 12.3 19.2 27.3</th>
</tr>
</thead>
</table>

The critical depth over the crest, corresponding to a discharge \( Q = 20 m^3/sec \), is \( (z_a - f_a) = 0.82 \text{ m} \), and the specific head is:

\[ [(z_a - f_a) + \Omega_a / 2b] = (0.82 + 7.457/2 \times 10.19) = 1.19 \text{ m} \]

Initially, on completion of the weir, since the velocity of the water upstream of the weir is very low, the specific head will be identical to the head of water, that is, with the difference \( (z_0 - f_0) \) in level between the water upstream and the crest, the flow immediately upstream of the weir will be certainly
subcritical, and the transition from the supercritical flow immediately at the structure, is attained by the formation of a hydraulic jump [54].

When accretion upstream reaches cill level so that the bed slope finally becomes $10\%$ as expected, the discharge $Q = 20 \text{ m}^3/\text{sec}$ will correspond to a depth $y_u$ in conditions of uniform flow:

$$y_u = \left( \frac{Q}{C l^{1/2}} \right)^{3/5} = \left( \frac{20}{30 \times 20 \times 0.01^{1/2}} \right)^{3/5} = 0.52 \text{ m},$$

which is lower than the critical velocity

$$\sqrt{gy_u} = \sqrt{9.81 \times 0.52} = 2.26 \text{ m/sec},$$

and with a specific head $H_u$

$$H_u = y_u + \frac{V_u^2}{2g} = 0.71 \text{ m}$$

Immediately above the weir, the river is nearly rectangular, 25 m wide. When it has been filled by deposition to the level of the crest, the specific head is nearly equal to the critical head as computed, (1.19 m), to which a depth of $(z_0 - f_0) = (z_0 - f_u) = 1.16 \text{ m}$ corresponds.
The transition from this depth to the depth of the subcritical flow in the channel upstream is represented by the backwater curve.

B) Stilling pool

The stilling pool is not paved, and is similar to the type described in chapter II para. 3.1. The depth of water \((Z_3 - I_3)\) as computed, is 0.355 m.

The maximum depth of scour at the toe of the weir is found by the Schocklitsch formula (5):

\[
Z_0 - Z_3 = 5.25 + 1.19 - 0.36 = 6.08 \text{ m,}
\]

\[
q = \frac{20}{8 + \frac{4}{3} \times 0.82} = 2.2 \text{ m}^3/\text{sec m}
\]

\[d_1 = 160 \text{ mm.}
\]

Therefore:

\[Z_3 - f_h = 2.11 \text{ m}
\]

\[f_3 - f_h = 2.11 - 0.36 = 1.75 \text{ m.}
\]

The 2.0 m deep foundation and the sheet piling at the toe protect the structure undermining. The distance \(X\) of the cascade from the toe of the weir is approximately:

\[X \approx \sqrt{2(Z_g - f_h)(Z_g - b_3)} \approx \sqrt{2 \times 0.82 \times (0.82 + 5.25)} = 3.16.
\]

Level \(z_e\) can be computed assuming the total head is constant between a section at \(Z_g\) (where the velocity head can be neglected) and a section at \(Z_3\). Therefore:

\[z_e - f_3 = (Z_3 - f_3) + V_e^2/(2g) = 0.36 + 0.40 = 0.76 \text{ m.}
\]

C) Check on stability and resistance at a section through the crest.

Referring to fig. 63. The weir was seven courses of 1 m gabions high. The crest was lined with concrete and the gabion foundation was restrained by a reinforced concrete beam connected to reinforced concrete sheet piling 0.50 m thick.

This particular type of structure is not comparable to the gravity structures described in para. 6 chapter II. Resistance to sliding was achieved by the combination of the sheet piling, and friction between the base and the foundation soil.

The stability against overturning of the part of structure above section \(A-B\) has to be verified; forces acting on the course of gabions below section \(AB\) are neglected.

The following values are assumed:

- **gabions**:
  - \(\gamma_s = 2600 \text{ kg/m}^3\)
  - \(n = 0.30\)
  - \(\gamma_s(1 - n) = 1820 \text{ kg/m}^3\)
  - \(\gamma_{aw} = (\gamma_s - \gamma_w)(1 - n) = 1120 \text{ kg/m}^3\)

- **water**:
  - \(\gamma_w = 1000 \text{ kg/m}^3\)

- **soil**:
  - \(\gamma_s = 2600 \text{ kg/m}^3\)
  - \(n = 0.35\)
  - \(\gamma_{aw} = (\gamma_s - \gamma_w)(1 - n) = 1040 \text{ kg/m}^3\)

For the worst conditions, the values of forces and their corresponding moments taken about the hinge point \(A\), are tabulated below:

<table>
<thead>
<tr>
<th>Forces</th>
<th>Moments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of gabion structure</td>
<td>30820 kg</td>
</tr>
<tr>
<td>Weight of water on the crest and on the upstream backfill</td>
<td>3690 kg</td>
</tr>
<tr>
<td>Weight of the upstream backfill</td>
<td>9100 kg</td>
</tr>
<tr>
<td>Pressure of water upstream ((H_{uw}))</td>
<td>25140 kg</td>
</tr>
<tr>
<td>Pressure of water downstream ((H_{uw}))</td>
<td>290 kg</td>
</tr>
<tr>
<td>Thrust of soil upstream ((H_{us}))</td>
<td>6240 kg</td>
</tr>
</tbody>
</table>

The coefficient of stability is therefore:

\[s_r = \frac{50880 + 8300 + 31280 + 70}{48800 + 12480} = 1.5
\]

The study of pressure distribution for section \(A-B\) shows a maximum pressure, at a point \(A\), which does not exceed the safe bearing capacity of the soil. The resultant of forces passes outside the middle third of the base section, at a distance from \(A\) equal to:

\[X_A = \frac{M_{S} - M_{SH}}{\Sigma V} =
\]

\[= (50880 + 8300 + 31290) - (48800 + 12480 - 70)\]

\[= 0.67 \text{ m}
\]
Assuming that the gabion structure will not resist tensile stress, $\sigma_A$ is:

$$\sigma_A = 2 \frac{\Sigma}{3 \times XA} = 2 \frac{30820 + 3690 + 9100}{3 \times 67 \times 100} = 2.17 \text{ kg/cm}^2$$

D) Study of solid transportation

A survey made two years after the completion (1970), provided useful information concerning the total volume of material transported by the torrent.

Fig. 64 shows the natural longitudinal profile and that afterwards achieved upstream of the second weir.

The survey carried out in June 1973 recorded a volume of deposited material of about 12800 m$^3$. If this is related to the 6 sq km of catchment area, the volume is equivalent to 1,070.00 cu m/sq km per year which coincides with the values published by the Department of the Italian Hydrographic Service in Bologna [55].

The data obtained from the survey are tabled below:

<table>
<thead>
<tr>
<th>Section</th>
<th>Reduced level of natural river bed</th>
<th>Reduced level of deposited material</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>215.06</td>
<td>220.06</td>
<td>8650</td>
</tr>
<tr>
<td>2</td>
<td>215.78</td>
<td>220.29</td>
<td>2470</td>
</tr>
<tr>
<td>3</td>
<td>217.44</td>
<td>221.00</td>
<td>640</td>
</tr>
<tr>
<td>4</td>
<td>217.96</td>
<td>221.16</td>
<td>630</td>
</tr>
<tr>
<td>5</td>
<td>219.74</td>
<td>222.80</td>
<td>324</td>
</tr>
<tr>
<td>6</td>
<td>220.92</td>
<td>222.96</td>
<td>94</td>
</tr>
<tr>
<td>7</td>
<td>223.77</td>
<td>224.57</td>
<td>—</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td></td>
<td></td>
<td><strong>12808 m$^3$</strong></td>
</tr>
</tbody>
</table>

The deposition of 1070 cu m/sq km was considered to be principally due to the transported bed load to which, a quantity of about 800 cu m/sq km per year of material carried in suspension and not deposited, must be added. The total volume carried by the Rio Canilio is therefore 1870 cu m/sq km per year.

b) Italy, Pesaro, Weir on the River Cesano - 1978.

The River Cesano, located between the districts of Ancona and Pesaro in Italy, has virtually no flow in summer but experiences high peak discharges during the rainy season. It flows through clay soils and almost every year these high discharges cause deep scour in the upper reaches of the river.

In 1978, a river bed stabilisation weir was constructed to protect the foundations of a bridge near Corinaldo.

A vertical weir with trapezoidal crest, with an unlined stilling pool was designed to regrade the bed from a natural 0.5% to a 0.3% slope (figs. 65, 66).

The design discharge was taken as $Q = 800$ cumecs. The depth of water flowing over the crest (see fig. 66) is approximately 2.70 m ($^*$); this value was obtained by considering the flow over the crest $Q_c = \Omega_g \sqrt{\frac{g \Delta h}{b}}$, where $\Omega_g$ is the area of cross section of the critical flow, in sq m, $b$ is the corresponding width of water surface, in m, and $g$ is the acceleration due to gravity:

$$\begin{array}{cccc}
(z - f_0) [m] & \Omega_g [m^2] & Q_c [m^3/sec] & (H_c - f_0) [m] \\
1 & 41.0 & 126 & 1.48 \\
1.5 & 73.8 & 283 & 2.25 \\
2.0 & 107.0 & 474 & 3.00 \\
2.5 & 140.8 & 697 & 3.75 \\
3.0 & 175.0 & 950 & 4.50 \\
\end{array}$$

($^*$) In the hypothesis of a non-submerged weir, as verified later.

Fig. 64 - Material deposited upstream of the weir.
The specific head on the crest, corresponding to the design discharge 800 cumecs, is \( H_t-f_b = (z_g-f_b) + \left( \frac{Q}{\Omega} \right)^2/z_g = 4.05 \) m, and the upstream depth in the river is approx 3.50 m above crest; the wings have consequently a free board of approx half a meter.

Downstream of the weir, the depth of water of uniform flow is found as for a wide rectangular channel:

\[
(z_3 - f_b) = \left( \frac{Q}{C l^{1/2} l} \right)^{3/5} = \left( \frac{800}{33 \times 0.005^{1/2} \times 68} \right)^{3/5} = 2.64 \text{ m}
\]

where: \( l = 68.0 \text{ m} \) is the average width of the river, \( i = 0.05 \) its slope and \( C = 33 \text{ m}^{-1/3} \text{ sec}^{-1} \) (assumed roughness coefficient). The velocity for uniform flow is:

\[
V_3 = \frac{Q}{(z_3-f_b) l} = \frac{800}{2.64 \times 68} = 4.46 \text{ m/sec}
\]

The flow is subcritical, as \( \frac{V_3^2}{g(z_3-f_b)} = 0.77 < 1 \).

As the downstream depth is much lower than the limit of submergence, the crest is not submerged.

The maximum depth of scour is calculated by the Schoc-klitsch equation, considering the crest as rectangular:

\[
z_3 - f_b = 4.75 \left( \frac{z_0 - z_3}{d_{90}} \right)^{0.57} = 4.75 \left( \frac{2.86}{140^{0.32}} \right)^{0.57} = 4.94 \text{ m}
\]

(where \( z_0 - z_3 = 3.50 + 2.00 - 2.64 = 2.86 \text{ m} \) and \( d_{90} = 140 \text{ mm} \)); therefore

\[
f_b - f_3 = 4.94 - 2.64 = 2.30 \text{ m}.
\]

It is not necessary to show the detailed calculations for stability checking, as they would be as recommended in para. 6, chapter II, and as applied in the paragraph III(a). The only variation suggested in the present case is to take the elevation \( z_e \) as equal to \( z_3 \).

c) Australia, Hume Highway at Gundagai - 1978.
Vertical weir; the body of the structure is built of gabions, the upstream apron, the crest and wings with Reno mattress. The weir was erected to protect the piers of the bridge from scour (fig. 67).

Weirs on the Togodolé torrent (fig. 68).
c) **Italy, Castel dell'Alpe (Bologna) - 1975.**
Weir at the mouth of the torrent Savena on the Castel dell'Alpe lake (fig. 69).

d) **Italy, Trebisacce (Cosenza) - 1979.**
Weir built for erosion control on a torrent carrying heavy boulders. The sill is lined with concrete (fig. 70).

g) **Italy, S. Leo (Pesaro) - 1979.**
Structures protecting the base of the ancient castle of S. Leo; a series of weirs and deep gabion drainage counterforts were used to stabilize the slope (fig. 71).

Weir on the Potomac river, to maintain a constant water depth in the river, for pumping into the Washington aqueduct during dry weather flow.

The gabions were filled at a distance from the site and placed by helicopter. Filter cloth was used inside the baskets to increase the rate of silt deposition.

The novelty of the structure and its method of construction attracted considerable attention and was reported in the United States press (figs. 72, 73, 74, 75).
i) Yugoslavia, Pisino (Istria) - 1975.
Weir on the Vranja torrent, tributary of the River Bogliuncica. The crest was lined with concrete (figs. 76, 77).

Series of small weirs for the control of erosion. The crests were lined with concrete (figs. 78, 79).

m) Iraq (Hemrem) - 1977.
Temporary gabion weir constructed for the protection of the central part of the clay core of a big earthfill dam under construction.
The weir was flooded by a one hundred years discharge of 4650 cumecs, with 6 m height of water, without suffering appreciable damage (fig. 80).
1.1.2 With counterweir

a) **Indonesia, Bogor Java - 1977.**

Weir and counterweir in the Ciapus Project (fig. 81).

b) **Republic of Cabo Verde, S. Tiago - 1977/78.**

Weirs for erosion control. These structures are part of an important river training and land reclamation project (figs. 82, 83).
1.2 Stilling pool with lined floor

1.2.1 By simple apron

   The permeability of the structure is clearly demonstrated (fig. 84).

b) Australia, Benalla (Victoria).
   Small gabion weir. Again the permeability of the structure is evident. Upstream, the banks have been lined with Reno mattress and gabions. Downstream the side walls are entirely in gabions (fig. 85).

c) Barbados, St. Andrew.
   Series of weirs for soil conservation.
   Photograph 86 shows a general view of the structures immediately after construction. Photograph 87 shows the good result of the training, when vegetation has grown around and through the structures themselves (figs. 86, 87).
Weir for flood control at S. Caterina. As the horizontal surface of the steps is very narrow, the structure can be considered as a vertical weir (fig. 88).

Gabion drop structure. The downstream face is slightly battered to improve the stability of the structure (fig. 89).

f) Brazil, Miguel Pereira (Rio de Janeiro) - 1979.
Series of weirs for erosion control.
The upstream face, the foundation and the wings were lined with a synthetic filter cloth to prevent the fine particles of the foundation soil from migrating through the structure (figs. 90, 91).
1.2.2 Jump control by broad crested counter weir

a) Yugoslavia, Pisino (Istria) - 1974.
The trapezoidal crest has its central portion depressed, to concentrate low discharge in the centre and to prevent channel formation close to the banks (fig. 92).

2 Stepped weirs

2.1 Stilling pool with unlined floor

Small weir in a canalized water course, near Highway No. 401 (fig. 93).
2.2 Stilling pool with lined floor

2.2.1 With simple apron

a) Libya, Benghazi - 1970.
Weirs erected on Wadi Ghot Sultan River (fig. 94) and on its tributaries Wadi Bu Sanab (fig. 95) and Wadi Bakur (fig. 96).

b) Malaysia - Pulau Langkawi - 1964.
Gabion spillway (fig. 97).
c) Thailand - Nau Province (Pua District) - 1978.
Huai Bua Weir: stepped weir, with downstream apron and lined banks. The structure is shown under construction: note that water was allowed to pass over it before it was completed (figs. 98, 99).

2.2.2 Jump control by abrupt rise

a) Australia, Dangenong Valley (Victoria) - 1975.
The weir is shown after completion and during spate conditions (figs. 100, 101).

b) Libya, Beni Walid - 1976.
Weir for Wadi Mardum irrigation scheme: a series of structures was built with the purpose of retarding the flow velocity and thereby increasing seepage through the bed to recharge the aquifers below (figs. 102, 103).

c) Libya, Derna - 1975.
Stepped weir for erosion control erected on a tributary of the Wadi Derna (fig. 104).

d) Libya, Tripoli - 1972.
Weir on Wadi Megenin (fig. 105).
2.3 Cascades

a) Canada.
Gabion stepped structure built for the Canadian National Railways (fig. 60).

b) U. K., Lancashire - 1972.
A cascade on a stream diversion near the M62 Motorway. The wire mesh of the gabions was galvanised and further protected with a PVC coating (fig. 107).

c) South Africa.
A cascade at the outlet of a culvert. The growth of vegetation indicates the success of the scheme (figs. 108, 109).
d) **South Africa.**
Series of weir for soil conservation (fig. 110).

c) **Switzerland, Altdorf - 1979.**
Training scheme to safeguard a village downstream (fig. 111).

f) **New Zealand - South Canterbury.**
Structures at the toe of a slip area near Mackenzie Pass in the Ophii River Basin. The soil conservation project was quite successful, as the growth of vegetation shows (fig. 112).

g) **Italy, Castel dell'Alpe (Bologna).**
Training on the Savena torrent draining from Lake Castel dell'Alpe (1976).

On February 1951, an extensive landslide occurred Southwest of the village of Castel dell'Alpe (Bologna), and completely obstructed the torrent Savena (fig. 58).

A lake was formed, about 1 km long and 19 m deep, with a volume of about 1 million cu.m.

To avoid further slips of the side slopes which were continuously eroded (average bed slope = 15%), an initial training programme was carried out with some stepped gabion weirs, (see figs. 15, 16, chap. I).

These structures provided the required protection, worked well, and passed discharges greater than calculated (110 cumecs).

During the disastrous flood in that part of Italy in November 1966, some of the existing structures were undermined by the exceptional flow. Gabion structures were again used for the remedial works for which they were particularly suited for speed of construction and their adaptability to settlement.
The new river bed was cut directly through the landslide debris and protected by a series of sloped weirs, their downstream faces being grouted with sand asphalt mastic. The weirs function as a cascade: the energy being dissipated by a series of hydraulic jumps, as described below, see fig. 113 [36], [37].

The design allowed for a discharge of 150 cumecs from a catchment area of 21 sq km.

Fig. 114 shows the longitudinal profile of the river bed and the positions of the weirs; these were 2.50, 3.00, 4.00 or 6.70 m high.

Fig. 113 - Section through a typical cascade.
Fig. 114 - Savena training scheme.
Calculations for a weir having similar characteristics to those in the project are set out below. The height of the structures under consideration is 4 m, other dimensions are shown in figs. 115, 116 and 117.

The uppermost part of the downstream face of the weir is curved according to the Creager-Scimemi profile, represented by equation \( y = 0.47x^{1.8} \) and designed for a 2 m water head on the crest. This dissolves into a 45° slope to form the central portion which changes to a circular arc for the lowest section joining the stilling pool. \( x \) and \( y \) represent the ratios between the co-ordinates \( X \) and \( Y \) of the profile (see fig. 117), and the head of water at the crest.

\( z_0 \) is computed using equation (3), valid for broad crested weirs:

\[
Q = \mu l_{w}(z_0 - f_0)^{2/3}2g(z_0 - f_0)
\]

where \( l_{w} = 20.000 \text{ m} \), \( \mu = 0.64(*) \):

\[
z_0 - f_0 = \left( \frac{Q}{\mu l_{w}^{2/3}2g} \right)^{2/3} = \left( \frac{150}{0.64 \times 20.00 \times \sqrt[3]{2 \times 9.81}} \right)^{2/3} = 1.91 \text{ m}
\]

The velocity head corresponding to the subcritical flow upstream of the weir, is therefore:

\[
V_0^2/2g = Q^2/2g l_{w}^2(z_0 - f_0)^2 = 150^2/2 \times 9.81 \times 24^2 \times (1.91 + 1.40)^2 = 0.18 \text{ m}
\]

where \( l_{w} = 24.00 \text{ m} \) is the width of the cross section of the river. The total head, referred to level \( f_0 \), is:

\[
(z_0 - f_0) + V_0^2/2g = 1.91 + 0.18 = 2.09 \text{ m}
\]

(*) The low value of ratio \((f_0 - f_0)/(z_0 - f_0)\) allows a high discharge coefficient to be chosen. See [37].

The dimensions of the stilling pool are calculated as in chapter III. The related initial depth \( z_1 - f_0 \) at the toe of the slope is obtained by solving equation (8) through a trial and error procedure, neglecting energy losses between section 0 and 1:

\[
z_1 - f_0 = 0.54 \text{ m}(**)
\]

The related obtained depth is given by equation (10):

\[
(z_2 - f_0) = \frac{(z_1 - f_0)^2}{2} + \frac{2Q^2}{\sqrt{g l_{w}^2(z_1 - f_0)}} + \frac{(z_1 - f_0)^2}{4} = -0.54 + \frac{2 \times 150^2}{9.81 \times 24^2 \times 0.54} + \frac{0.54^2}{4} = 3.30 \text{ m}
\]

(**) If equation (9) is used instead of equation (8), the result is \( (z_1 - f_0) = 0.52 \text{ m} \).
PLAN

CROSS-SECTION AA
1: Creager-Scimemi profile.
2: Gabions sealed by sand asphalt mastic
3: Side walls
4: Concrete wings
5: Concrete slabs
6: Sand asphalt
7: Original slope

CROSS-SECTION BB

75
The depth is dictated by the weir positioned immediately downstream; its crest being 1.40 m above the upstream pool. The weir produces a head of \((1.40 + 1.91) = 3.31\) m above the pool itself.

The minimum length of the stilling pool, to avoid the jump moving downstream is:

\[ 6.9(z_2 - z_1) = 6.9(3.30 - 0.54) \approx 19.00 \text{ m} \]

On this and the previous page structures erected after 1966 are described (see figs. 118, 119, 120, 121, 122).

The drawings and photographs show that in each case the body of the structure is entirely formed of gabions on a foundation of asphaltic concrete. The crest is formed by a concrete cap, shaped according to the required profile, while the downstream slope is faced with 0.5 m gabions, completely sealed with 300 kg/sq m of sand asphalt mastic.

The stilling pool floor is lined with concrete blocks keyed together with steel cramps. The side walls are in gabions grouted with sand asphalt mastic. The width of the weir and side wall foundations, combined with the draining capacity of the structure and the impermeable external surface, provide the necessary stability and loading compatible with bearing capacity of the soil.
3 Sloped weirs

3.1 Stilling pool with unlined floor

a) Italy, Rubiera (Modena).
Flood channel inlet on the river Secchia.
The slope has been lined with gabions sealed by sand asphalt mastic (fig. 123).

3.2 Stilling pool with lined floor

3.2.1 Jump control by broad crested counter weir

Such a flood occurred in the Veneto region, where the depth of water in the rivers exceeded all previous records. At Bassano-on-Brenta, a maximum depth of 5.60 m was measured, against a former maximum value of 4.75 m recorded on 16.9.1882. The corresponding discharge, evaluated at Bassano, was estimated as about 2800 cumeecs.

This flood caused damage along the river and, in particular, the collapse of a concrete weir protecting the Carturo bridge near Padova, whose piers were scoured out to a depth of about 3 metres (fig. 124). The collapse of the weir brought severe erosion to a long reach of the river upstream of the weir.

In order to restore the hydraulic regime of the River Brenta, which had been drastically altered by the flood, and to protect the piers of an important bridge (a 15 span structure), a new weir was constructed.

(*) This paragraph has been prepared with the assistance of Dr. Emilio Baroncini - Technical Director in the Ministry of Public Works, Head of Operation of Water Authority of Padova and Este and Director of the Laboratory of Hydraulic Models of Volta Barozzo (Padova), who was both designer and site engineer for this project.
A sloped gabion weir, sealed by sand asphalt mastic (fig. 125) was chosen.

B) Hydraulic calculations

B.1) Hydrology

The new weir and the whole regrading of the bed downstream of the bridge were calculated assuming a maximum discharge of 2800 cumecs (*) as estimated by the Hydrographic Service.

The hydraulic behaviour of the weir was to be verified also for medium discharges and therefore a hydrological study to determine the graph of flood duration was carried out. This study was carried out, using the Gumbel method, taking into account the maximum discharges recorded in the last 44 years by the Hydrographic Service.

The results of the study are shown at fig. 126 where, for different values of the water depth (cm), values of \( y \) (Gumbel equation), and of the return period \( T \), (years) are shown, and the corresponding discharges.

According to this study, the discharge of 2800 cumecs has a return period of several hundreds of years.

B.2) Stable slope

The weir was positioned in such a way as to guarantee the protection of the piers even during the maximum flood. The level of the crest was fixed at 26.65 m above sea level so as to obtain, by accretion, a river bed level at the foundation of the bridge of 26.80 m above sea level.

The stable slope was evaluated as 0.55 m/km; i.e., supposing the maximum discharge is flowing in conditions of uniform flow:

\[
i_c = \frac{(v_u)^{10/3} B^{4/3}}{C^2 Q^{4/3}} = \frac{(v_u)^2 n^2}{R^{4/3}} = 0.00055
\]

where:

- \( Q = 2800 \) cumecs: design discharge;
- \( u_i = 1.88 \) m/sec: permissible velocity, corresponding to an average size of river bed materials of 0.25 m;
- \( C = \frac{1}{n} \approx 35 \) m\(^{1/3}\) sec\(^{-1}\): roughness coefficient (Strickler), estimated according to the hydraulic analysis executed after the flood of November 1966;
- \( B = 210 \) m: wetted perimeter of the average cross section of uniform flow, approximately equal to the width of the river.

B.3) Hydraulic behaviour of the weir

In the section under study the flow is subcritical, the natural river slope being much flatter than the critical slope. This means that the water profile must be studied commencing at the downstream end and proceeding upstream, and was in fact started from a section positioned 300 m downstream of the structure, where the river is perceptibly narrower.

The conditions of flow in this section have been taken as initial conditions for the study of water profiles: width of the river (supposed rectangular) \( l = 138 \) m; elevation of river bed 23.25 m above sea level; elevation of water surface 28.90 m a.s.l.; depth of water (28.90 – 23.25) = 5.65 m.

In such conditions of flow, the coefficient of roughness (Strickler) is:

\[
n = \frac{R^{2/3} i_c^{1/2} \Omega}{Q} = \frac{5.23^{2/3} \times 0.00135^{1/2} \times 780}{2800} = 0.0308 \text{ m}^{-1/3} \text{sec}
\]

i.e. \( C = 1/n = 32.4 \) m\(^{1/3}\) sec\(^{-1}\).

Starting from the above conditions, and studying the backwater profile of the barrage, for the maximum discharge of 2800 cumecs according to Ven Te Chow's "Standard Step Method" [20] an elevation of water surface \( z_3 = 29.10 \) m a.s.l. and a depth \((z_3 - f_3) = (29.10 - 23.35) = 5.75 \) m was obtained.

During the hydraulic calculation of the structure and its stilling pool, whose dimensions are shown in fig. 125, it was found that the worst conditions did not take place for the maximum discharge but at an intermediate stage. In fact, the maximum discharge does not flow over the crest and the counterweir in critical conditions; the flow always remains subcritical, while it becomes supercritical and forms the hydraulic jump at the toe of the spillway for moderate (and therefore more frequent) discharges.

The calculation to establish the distance of the jump from the toe of the spillway was carried out by the usual methods [20] and shows the following results:

<table>
<thead>
<tr>
<th>( Q ) (cumecs)</th>
<th>( \Delta x ) (m)</th>
<th>( F_r ) (Froude number)</th>
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<tr>
<td>550</td>
<td>13.50</td>
<td>4.83</td>
</tr>
<tr>
<td>850</td>
<td>22.50</td>
<td>3.76</td>
</tr>
<tr>
<td>1200</td>
<td>27.50</td>
<td>3.73</td>
</tr>
<tr>
<td>1400</td>
<td>28.50</td>
<td>3.55</td>
</tr>
<tr>
<td>1650</td>
<td>29.50</td>
<td>3.55</td>
</tr>
</tbody>
</table>

(*) Value corresponding to the maximum discharge measured (28 km) upstream, at Bassano, the lowest point of the mountain basin of the River Brenta.
Fig. 124 - Scour at the bridge piers.
Fig. 125 - Gabion sloped weir.
Fig. 126 - Hydrological study.
Fig. 127 - Model test.

Figs. 128, 129, 130 - Model test: the erosion at the right side (fig. 129) is trained by a downstream stone apron (fig. 130).
Figs. 131, 132, 133 - The weir under construction.
Fig. 134 - The structure after some years of life.

The distance $\Delta x$ is less than the length of the stilling pool, but the trend of values of the Froude number indicated the formation of an oscillating jump, not easily contained within a fixed pool length.

It was therefore advisable to employ two dissipation devices, a mastic grouted stilling pool and a layer of random stones downstream of the counter weir.

As a check on the behaviour of the whole scheme, which could not be easily evaluated by means of a mathematical model (due to the curved reach of the river and to the presence of irregularities in its bed), it was necessary to carry out several tests on a scale model, built at the laboratory of Volta Barozzo (Padua).
The tests confirmed the stilling pool was not sufficient for complete energy dissipation (see figs. 127, 128, 129 showing the erosion at the right side, downstream of the pool) and proved the effectiveness of the downstream stone apron (fig. 130).

After construction, the structure worked for some years according to the design requirements; only the unexpected morphological evolution of the river bed, subjected to a severe and general erosion, caused hydraulic problems. The lower water elevation downstream of the weir caused the jump to move downstream of the pool. The consequent erosion damaged part of the downstream gabion apron and stone protection. It was however easy to carry out maintenance works on that second part of the dissipation device; the stone layer was repaired and strengthened to such an extent as to make the structure able to work in the new hydraulic conditions.

B.4) Prevention of undermining

The length of the stilling pool, the presence of a piled cut off and upstream and downstream aprons guaranteed the safe behaviour of the structure against undermining. Checks were made using the Bligh's or Lane's formulae, and were largely acceptable: the difference between the upstream and downstream water surface is 26.65 - 23.35 = 3.30 m (maximum difference obtained for an intermediate rate of discharge), while the soil, of sand and gravel with intermediate clay layers, showed a sufficient stability against seepage, and its Bligh's coefficient can be taken as 15. Consequently, the minimum total length of path under the structure according to Bligh's formula is approximately 50 m, while the real length exceeds 100 m.

The study of seepage under the foundation of the weir was also carried out by the Institute of Hydraulics (University of
Padua), by means of a Hele-Shaw model.

From the flow net of seepage water the value of the hydraulic gradient in the most dangerous positions was found and was compared with the critical value of the soil, which depends on the specific gravity and the porosity of its components.

The coefficient of safety against undermining, given by the ratio between the natural critical gradient and that one found by calculations, was about 20; this is greatly in excess of the usual acceptable values.

C) Calculation of stability and resistance

All such checks have been carried out according to the usual methods, and gave acceptable values of the maximum foundation pressure. Both the coefficient of stability against sliding and uplift were acceptable. The latter was found by taking into account the results of the mathematical model mentioned above, which gave the distribution of uplift pressure along the whole foundation.

D) Construction

The structure was founded directly on the river bed, and was anchored by a double row of reinforced concrete piles, 10 m long and placed at 1 m centres (fig. 131).

The work was carried out in two stages: the right half was constructed first, diverting the river to the left. In this stage, the construction of the gabions proceeded quickly and without problems, employing mechanical filling methods.

As soon as a part of structure was finished, the sand asphalt mastic work commenced (fig. 132). The mastic was poured directly from the insulated tankers or, in the more difficult locations, by an excavator.
In order to allow a deep penetration and to obtain a stronger protection, the mastic was poured at a rate of 350-400 kg/m².

The left side was then constructed, diverting the flow to the right half, now ready for use (fig. 133). In this second phase, some difficulties were experienced in the construction of the gabions in the stilling pool, due to the increased upstream water head causing a greater flow of water to seep in to the work area.

The side walls were consolidated by sand asphalt mastic, in order to avoid seepage through the wings; the downstream and upstream cut-off piles were deeply keyed into the banks.

The construction of the weir started in September 1967 and ended in June 1970. In the first 6-7 years of life (fig. 134) no maintenance of the structure was required; later, some repair works to the protective mastic layer of the stilling pool at the toe of the spillway were necessary.

b) Italy, Sasso Marconi (Bologna) - 1966.

Sloped weir, erected by the Bologna City Water Supply Department.

After the gabion structure had completely settled, the crest, glacis and apron were clad with concrete (figs. 135, 136, 137).
3.2.2 Jump control by abrupt rise

a) Libya, Tripoli - 1972.
An example of a gabion sloped weir, the first to be erected in Libya, on the Wadi Megenin (fig. 138).

b) Libya, Bir Ayyad (Tripoli).
Gabion spillway for a sandfill dam. The dam consists of a central core of sand lined with gabions. Impermeable sealing was effected by membranes within the core and under the gabions (figs. 139, 140, 141).
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