Flow Over Embankment Gabion Weirs in Free Flow Conditions

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Abstract

Gabion weirs have been widely used in rivers restoration and diversion water projects because of their hydro-environmental aspects and eco-friendly features. In this study, a series of laboratory tests were performed to investigate the effects of side ramp slope, crest length, and porous media properties on the flow regimes, water-surface profiles, discharge coefficients, and energy dissipation in embankment gabion weirs with upstream and downstream slopes. 24 physical models of solid and gabion weirs with three different upstream/downstream slopes (90°, 45° and 26.5°) were created. For gabion weirs, three different filling materials were tested. To investigate the complexity of flow over the porous-fluid interface and through the porous material, three-dimensional (3D) numerical simulations were developed. The results show that decreasing
upstream slopes, from 90° to 26.5°, leads to decreased discharge coefficients. However, in all cases, gabion weirs lead to greater discharge coefficients than those of similar solid weirs. For milder side slopes, discharge ratios passing through all faces of the gabion weirs decreased nonlinearly. Moreover, with increasing the inlet discharge, relative energy dissipation was reduced up to 45% in gabion weirs.

Keywords: Embankment weir; Gabion weir; Energy dissipation; Discharge coefficient; Porous media

1. Introduction

Weirs are hydraulic structures that can be installed in open canals or rivers to regulate the flow discharge and upstream water level. Hydraulic structures are generally constructed with concrete, however, gabion structures are of great interest in hydraulic engineering works such as rivers erosion control structures, headwalls and culvert outlets, drop structures, and weirs. Apart from having a good hydraulic functionality, a gabion weir has crucial effects on the surrounding environment ecology. The permeable body of these structures allows water to seep through the porous body. Consequently, organic and inorganic substances can pass downstream through the pores. Through flow reduces the sedimentation behind the weir (Michioku et al. 2005; Salmasi and Abraham 2020a; Mohamed 2010). Moreover, bacteria inhabiting the pores of the gabion weir degrade organic matter in the water and results in purification of downstream water. In addition, the flow turbulence within the permeable body increases the aerobic degradation of organic materials. Thus, the gabion weir can be an alternative to a concrete weir due to minimal negative impacts on the ecological system (Mohammadpour et al. 2013). There are other reasons
to use these types of structures. For instance, Salmasi and Abraham (2020a) discussed the advantages of gabion weirs for reducing erosion, high flexibility against tensile and bending forces, durability, permeability, and cost-effectiveness. Gabion weirs in irrigation canals can reduce free board height and this reduces construction costs. Free board is the vertical distance between canal top and normal depth of water in canal.

Traditional weirs are solid (impermeable) and flow can only pass over these weirs but with gabion weirs, flow can pass through interstitial spaces as well. This increases canal conveyance capacity and thus reduces free board.

Kells (1993) conducted experimental tests on porous weirs and reported that 25-50% of inlet discharge can pass through the porous body. Michioku et al. (2005) studied the flow regime over a rubble mound weir under submerged, transitional, and full overtopping conditions. They proposed a functional relationship between flow capacity, porosity, particle diameter, and geometry of rubble mound weirs. Leu et al. (2008) experimentally tested time-averaged velocity and turbulent velocity components, turbulent kinetic energy, and Reynolds stresses for the broad-crested weirs with porosities of 0, 34.9, and 47.5%. Measurements indicated that the turbulence intensity, turbulent kinetic energy, and Reynolds shear stresses above the porous weir decrease with increasing the porosity and grain diameter. Mohamed (2010) performed an experimental study to estimate the discharge over rectangular rigid and gabion weirs under both free and submerged flow conditions. Mohamed (2010) presented non-dimensional relationships for discharge coefficients that were based on laboratory tests. Mohammadpour et al. (2013) studied different turbulence models on predictions of the flow field over porous broad-crested weirs. In order to verify the numerical simulations, the experimental results of Leu et al. (2008) were used. Mohammadpour et al. (2013) indicated that the standard $k-\varepsilon$ and the Reynolds stress model (RSM)
provide accurate results for estimating the water surface profile around the gabion weirs. Salmasi et al. (2011) developed empirical equations for calculating the discharge coefficients over porous rectangular weirs under free and submerged flow conditions.

More recently, Safarzadeh and Mohajeri (2018) applied a $k-\varepsilon$ model to determine the hydrodynamics of rectangular broad-crested porous weirs. According to their three-dimensional numerical simulations, the drag coefficient of the nonlinear term in the Forchheimer model was calibrated and empirical equations were developed for computing discharge coefficients. Salmasi and Abraham (2020a) provided a discussion on the study of Safarzadeh and Mohajeri (2018); they found a reasonable agreement between the reported formula of Hager and Schwalt (1994); Govinda Rao and Muralidhar (1963) and the numerical simulations of Safarzadeh and Mohajeri (2018). They also introduced a method for estimating hydraulic conductivity in gabion weirs.

A series of laboratory tests were conducted to determine the discharge coefficients for gabion rectangular weirs in free and submerged conditions by Salmasi et al. (2020b). Their observations indicated that the discharge coefficients of gabion weirs in free flow conditions was 17.2% greater than those for submerged flow conditions. Furthermore, as a result, Salmasi et al. (2020b) developed multivariable nonlinear regression equations to determine the discharge coefficients for gabion rectangular weirs in both free and submerged conditions.

The previously mentioned studies focused on solid and gabion rectangular weirs. The incorporation of a crump weir with a rectangular broad-crest weir represented a new configuration that is sometimes referred to in the literature as a “trapezoidal broad-crested weir” or an “embankment weir with upstream and downstream slopes”. The main advantages of trapezoidal broad-crested weirs include excellent stability, better hydraulic characteristics, and discharge efficiency. Earlier studies, as Tracy (1957), Isaacs (1981), and Hager and Schwalt (1994) focused
on revealing the influence of the geometries of rigid broad-crested weirs with upstream and/or downstream ramps on the flow pattern. Fritz and Hager (1998) founded that discharge capacity of a trapezoidal weir with 1V:2H slopes was 15% greater than that of a rectangular broad-crested weir.

Also, Sargison and Percy (2009) studied the flow of water over a solid embankment weir with 1V:2H, 1V:1H and vertical slopes in various combinations on the weir upstream and downstream faces. Hakim and Azimi (2017) conducted a series of laboratory tests to investigate the hydraulics of crump weirs and trapezoidal weirs with finite-crest lengths under the submerged-flow condition. They developed an empirical model to predict the discharge reduction factor based on the submergence level. Fathi-moghaddam et al. (2018) compared the hydraulic parameters of gabion crump weirs with gabion broad-crested weirs. They showed that the upstream slope of crump weirs and downstream slope of embankment gabion weirs had more significant effect on the discharge coefficients.

Apart from experimental studies, computational fluid dynamics is an important method to investigate hydraulic problems and has been applied to determine the flow characteristics over solid trapezoidal weirs. Sarker and Rhodes (2004) applied the standard k-ε turbulence closure model to simulate the flow over a broad-crested weir. They found that the computed free-surface profiles based on the volume-of-fluid (VOF) method was in good agreement with measured results. Kirkgoz et al. (2008) used the standard k-ε and standard k-ω turbulence models in 2D simulations of free-surface flows interacting with rectangular and triangular weirs. Comparisons showed that results from the standard k-ω turbulence model were in better agreement with measured values. Haun et al. (2011) applied Flow 3D and SSIIM2 to calculate the discharge coefficient over a trapezoidal broad-crested weir and Nourani et al. (2021) numerically estimated
the discharge coefficients \( (C_d) \) over the broad-crested weirs with different configurations. In addition, they utilized two intelligent models ANN, GPR and the hybrid models ANN-HHO, GPR-HHO to determine the discharge coefficients as accurately as possible. It was shown that ANN and GPR models can be used to generate a better estimate of the discharge coefficients of broad-crested weirs by using hybrid artificial neural network and Gaussian process regression with Harris Hawks optimization.

Many studies, including those referenced above, have considered flow over rectangular gabion weirs, while few studies have investigated flow over the embankment gabion weirs. Hence, the present study focuses on the effects of upstream and downstream slopes of weirs on water surface patterns, discharge coefficient values, and energy dissipation.

Here, three-dimensional numerical simulations were performed using the FLOW-3D package. These simulations will be used to investigate the flow behavior around and inside the gabion weirs.

2. Material and methods

2.1. Experimental layout

Experiments were carried out in the hydraulic laboratory of the Faculty of Agriculture, University of Tabriz, Iran, in a rectangular flume with 10 m length, 0.25 m width and 0.5 m height (Fig. 1).
Fig. 1 Scheme of water supply system

The slope of the flume was constant and equal to 0.002. The flow discharge was measured using a calibrated V-notch weir installed in the outlet of the flume with ± 1% accuracy. Water-level profiles for each model were recorded at five different discharges using a point gauge with ± 0.1 mm of reading accuracy. At low discharge, photographs taken from the flow through the body of weir during the experiments were used to extract the seepage flow surface profiles.

24 physical models comprising solid and gabion weirs with a 0.25 m height and with 0.25 and 0.5 m crest lengths were built. Side slopes of 45° and 26.5° were added to the upstream and the downstream faces of the rectangular weirs. As a result, four different embankment weirs and two rectangular models were created. The embankment weirs were constructed from the upstream triangle section, the rectangular part, and the downstream triangular section. A metal woven mesh with apertures smaller than the minimum size of the filling material was utilized to build the gabion weirs. Nevertheless, the meshes had negligible resistance against the flow (Fig. 2). Figure (3) shows the natural aggregates that were used in the gabion baskets.
As already discussed, the gabion weirs were made using natural aggregates with three different particle diameters and porosities. In order to determine the particle sizes, standard particle size tests were used. The distribution of the particles diameters was uniform with standard deviation of particle diameters such that $(\sqrt{d_{84}/d_{16}})$ equal to 1.32, 1.17 and 1.08 for mean diameters equal to
11.4, 10.6 and 6.7 mm, respectively. The solid weirs were made using a white PVC and the range of test conditions are listed in Table 1. There, \( W \) is the width perpendicular to flow direction, \( d_m \) is the average grain size in the gabion basket, and \( n \) is the porosity of the gabion. Seepage flow conditions refer to experimental tests of gabion weirs with low discharge so that flow passes only through the gabion weirs and there is no overflow discharge. Meanwhile, the overflow condition in Table 1 corresponds to higher discharge that creates both through-flow and overflow. The prefixes GP1, GP2 and GP3 refer to 11.4, 10.6, and 6.7 mm diameter stone particles. Figure (4) provides granulation curves for three different filled-sand gabions.

Table 1 Range of experimental variables

| W (cm) | Upstream/downstream slope (°) | \( d_m \) (mm) | \( n \) % | Prefix | Seepage flow condition | Overflow condition |
|--------|-------------------------------|----------------|-------|--------|-------------------------|-------------------|
| 25     | 90°                           | 6.7            | 38    | GP 3   |                         |                   |
|        | 45°                           | 10.6           | 40    | GP 2   |                         |                   |
|        | 26.5°                         | 11.4           | 45    | GP 1   |                         | \( Q = 11 - 32 \) (l/s) |
|        | Solid                         | -              | S     | S      |                         | \( H/L = 0.12 - 0.6 \) |
| 50     | 90°                           | 6.7            | 38    | GP 3   |                         |                   |
|        | 45°                           | 10.6           | 40    | GP 2   |                         |                   |
|        | 26.5°                         | 11.4           | 45    | GP 1   |                         |                   |
|        | Solid                         | -              | S     | S      |                         |                   |
2.2. Theoretical assessment and dimensional analysis

Calculation of discharge for flows over a broad-crested weir can be achieved using the continuity equation and Bernoulli’s equations, which can be written as (Salmasi et al., 2021):

\[ Q = C_d \left[ \frac{2}{3} \left( \frac{2}{3} g \right)^{\frac{1}{2}} \right] W H_t^{1.5} \]  

(1)

where \( Q \) is the inlet flow discharge, \( g \) is the gravitational acceleration, \( W \) is the width of the weir, \( H_t \) is the upstream total head and \( C_d \) is the discharge coefficient. The total upstream head is generally attributed to the cumulative contribution of the approaching flow velocity head and the overflow head upstream of the weir. The velocity head component can be neglected with respect to small approach mean velocities. Therefore, it is a common practice to relate flow discharge to the upstream water head over the crest (\( H \)) measured at a distance 4\( H \) upstream of the weir. Experimental measurements indicate that the discharge versus total head relationship strongly depends on the weir geometry and discharge coefficient represents the geometrical effects and
hydraulic effects, including friction loss, possible flow curvature, and non-uniform velocity
distribution (Safarzadeh and Mohajeri 2018)

The physical relation for discharge is clarified by Buckingham’s theorem which represents non-
dimensional relationships by identifying the fundamental independent parameters. Complete
overtopping flow discharge over gabion weirs include many factors including the weir geometry,
hydraulic conditions, and porous medium properties. A set of essential parameters for an
embankment weir in a rectangular canal is:

\[ f_1(Q, H, L, P, W, d_m, g, \rho, \mu, n, \theta) = 0 \]  

(2)

where, \( f_1 \) =functional relationship, \( L = \) crest length, \( P = \) weir height, \( d_m = \) mean grain diameter of
the filling material, \( \rho = \) water density, \( \mu = \)dynamic water viscosity, \( n = \) the porosity of filling
material, and \( \theta = \)angle of the upstream/downstream slope of the weir. Note that the width of canal
\( (W) \) and the height of weir was kept constant for all tests. By using dimensional analysis, one can
write:

\[ C_d = f_2\left( \frac{H}{L}, \frac{d_m}{P}, R_e, \theta, n \right) \]  

(3)

where, \( f_2 \) is a functional symbol and \( R_e \) is the Reynolds number. The Reynolds number was
sufficiently high \( (R_e > 10^4) \) in all experimental tests, so the effects of viscosity could be neglected
and Froude similitude could be developed (Table 1). Likewise, Eq. (3) may be simplified into Eq.
(4):

\[ C_d = f_3\left( \frac{H}{L}, \frac{d_m}{P}, \theta, n \right) \]  

(4)

where \( f_3 \) = functional symbol. In this study, Eq. (4) is selected as the fundamental relationship for
estimating the discharge coefficients.
3. Governing Equations and Numerical Package

In order to simulate the flows, Flow 3D commercial CFD software was used to solve the continuity (Eq. 5) and the unsteady Reynolds-averaged Navier–Stokes (RANS) equations governing fluid motion (Eq. (6)) as:

\[
\nabla.[\rho \bar{U}] = 0 
\]

\[
\frac{\partial (\rho \bar{U})}{\partial t} + \nabla.\{\rho \bar{U} \bar{U}\} = -\nabla \bar{p} + [\nabla. (\bar{\tau} - \rho \bar{U}'\bar{U}')] + \bar{f}_b - \bar{d} 
\]

where, \( p \) is the pressure; \( U \) is the velocity, \( t \) is the time, \( f_b \) is the body forces acting on the control volume and \( \tau \) is the shear stress. The parameter \( d \) is the drag force which represents the resistance to flow in a porous medium. In Eqs. (5) and (6), over bars denote time averaging and \( \prime \) indicates the instantaneous fluctuations around a time average. Under the simplifying assumptions of steady, incompressible, laminar flow of a Newtonian fluid, Eqs. (5) and (6) can be written as:

\[
\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 
\]

\[
u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} = -\frac{1}{\rho} \frac{\partial p}{\partial x} + \varphi \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} \right) + f_x - d_x 
\]

\[
u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} = -\frac{1}{\rho} \frac{\partial p}{\partial y} + \varphi \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 v}{\partial z^2} \right) + f_y - d_y 
\]

\[
u \frac{\partial w}{\partial x} + v \frac{\partial w}{\partial y} + w \frac{\partial w}{\partial z} = -\frac{1}{\rho} \frac{\partial p}{\partial z} + \varphi \left( \frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} + \frac{\partial^2 w}{\partial z^2} \right) + f_z - d_z 
\]

In these equations, \( u, v \) and \( w \) are the velocities in \( x, y \) and \( z \) directions, respectively, and \( \varphi \) is the fluid kinematic viscosity. Moreover, \( f_x, f_y \) and \( f_z \) represent body forces and \( d_x, d_y \) and \( d_z \) are the drag force components in directions that are indicated by the subscripts.
At low Reynolds numbers, the Darcy equation describes the relationship between the macroscopic or Darcy velocity and the hydraulic gradient in a porous media \((i)\). However, Joy (1991) reported that Darcy's law was not valid for high flow velocity conditions. The Forchheimer’s equation has been most used by many researchers for high Reynolds numbers. The hydraulic gradient in Forchheimer’s equation is calculated as follows.

\[
i = aU + bU^2
\]  

(11)

where \(a\) and \(b\) are known as resistance coefficients which depend on fluid viscosity, material, and the flow regime. These coefficients are written

\[
a = A \frac{\mu(1-n)^2}{n^3}
\]  

(12)

\[
b = B \frac{\rho(1-n)}{n^3}
\]  

(13)

where, \(A\) and \(B\) are experimental coefficients and they can be estimated as follows.

\[
A = \frac{180}{d_m^2}
\]  

(14)

\[
B = \frac{\alpha}{d_m} = \frac{1.8 \text{ to } 4}{d_m}
\]  

(15)

In the numerator of Eq. (15), the range of 1.8 to 4 was defined for smooth to rough components. In the present study, an iterative process was used in order to select this number. Thus, the contribution of Darcian and non-Darcian losses in porous media can be expressed in terms of drag coefficient, as follows:

\[
F_d = -\frac{1}{\rho U} \nabla p = A \frac{\mu(1-n)^2}{\rho n^2} + B \frac{\mu(1-n)R_e}{\rho n d_m}
\]  

(16)
The above equations were solved using the Flow-3D CFD package. The standard $k-\varepsilon$ turbulence model was utilized in order to account for turbulence. Mohammadpour et al. (2013) have previously reported the proper performance of the $k-\varepsilon$ in simulations of porous media and this past success in part motivated our use.

A structured mesh domain was used to simulate the physical model. To capture the small variations in the free surface near the solid region, fine meshes were deployed near solid boundaries of weir to obtain more accurate results. The cell sizes near the solid boundary were selected in such a way that the criterion of wall unit distance ($Y^+$) ranged from 11.225 to 30 for upper limit. $Y^+$ is defined as

$$Y^+ = \frac{\rho u^* M}{\mu}$$  \hspace{1cm} (17)

where $\rho$ is the fluid density, $u^*$ is the shear velocity, $M$ is the normal distance from the solid surface of weir and $\mu$ is the dynamic viscosity of the fluid (Salaheldin and Imran 2004).

To ensure the flow becomes fully developed, a 3.5 m region was installed upstream and a 2.5 m region was installed after the gabion weir, downstream. At the inlet, outlet, and the top boundary, a prescribed mass flow, outflow conditions and symmetry conditions were used, respectively. The bed and side walls of flume were assigned no-slip wall boundary conditions (Fig. 5).
4. Results and discussions

4.1. Validation

In this study, the experimental results of the measured upstream head were used to calibrate the coefficient $B$ for each porous medium by considering different $\alpha$ values. Table 2 presents the results obtained during calibration runs. A suitable $\alpha$ value for each porous media was selected based on a comparison between measured and calculated upstream head values. Here a suitable value of $\alpha$ after iteration is denoted as $\alpha^*$. The resulting values of $\alpha^*$ for the three materials (GP, GP2, and GP3) were 2.5, 3.0, and 5.0, respectively. The value of $\alpha^*$ for the GP3 material was outside of the expected range. According to Table 2, a decrease in mean grain diameter and porosity increases the resistance of the porous materials.

Figure 6 shows a comparison between simulated and observed water surface profiles above the porous weirs; the comparison was made to validate the numerical results. As shown in Fig. 6, the estimated water profiles with the standard $k$-$\varepsilon$ model are in good agreement with the experimental
results. The excellent agreement between the numerical model and the experimental measurements confirmed the accuracy of the model.

![Graph showing variation of predicted water surface profiles for different α values over a gabion broad-crested weir.](image)

**Fig. 6** Variation of predicted water surface profiles for different $\alpha$ values over a gabion broad-crested weir with $L = 0.5$ m, GP1 and $Q = 21.4$ l/s

**Table 2** Calibration of $\alpha$ for different filling materials

| Model | $\alpha$ | B     | Measured upstream head (m) | Calculated upstream head (m) | $\alpha^*$ |
|-------|----------|-------|-----------------------------|------------------------------|------------|
| GP1   | 1.0      | 86.96 | 0.3748                      |                              |            |
|       | 1.2      | 104.35| 0.3768                      | 2.5                          |            |
|       | 1.8      | 156.52| 0.3792                      | 0.381                        |            |
|     |     |     |     |
|-----|-----|-----|-----|
| 2.5 | 217.39 | 0.3812 |
| 3.5 | 304.35 | 0.3821 |
| 1.0 | 94.34  | 0.3774 |
| 1.2 | 113.21 | 0.3783 |
| GP2 | 1.8   | 169.81 | 0.3806 | 0.385 | 3.0 |
| 3.0 | 283.02 | 0.3849 |
| 4.0 | 377.36 | 0.3861 |
| 1.1 | 156.25 | 0.3857 |
| 1.8 | 281.25 | 0.3877 |
| GP3 | 3.0   | 468.75 | 0.3886 | 0.393 | 5.0 |
| 4.0 | 625    | 0.3914 |
| 5.0 | 781.25 | 0.3931 |

**Note:** The value of $\alpha$ and $B$ are coefficients in Eq. (15) and $\alpha^*$ (last column) is the preferred of $\alpha$ after CFD validation.

### 4.2. Flow characteristics

For the gabion weirs, two typical flow regimes occur: seepage flow and complete overflow (Fig. 7). The seepage flow can be classified as either through flow with no-overtopping or through flow with partial overtopping. For the smallest discharge rates, water seeped through the permeable body of weir without any overtopping and no spatial variations were observed over the crest. The seepage flow profile infiltrated into the material at the upstream end and the curvature of water surface profile increased near downstream edge of weir. This caused small water jets to emerge.
from the downstream end of the gabion weir. With increasing discharge, partial overtopping flow occurred where some flow passed over the crest of the weir. The through flow with partial overtopping resulted in spatially varied flow with decreasing discharge (for the gabion weir). Along the crest length, the water entered into the permeable body and seeped through the gabion material. Comparisons of different upstream and downstream weir slopes showed that for similar flow rates and filling materials, the weirs with $\theta = 26.5^\circ$ had the highest upstream water level and seepage profile due to the higher resistance. Figure 8 shows overflow and through flow jets for the two cases identified by the annotations.

Fig. 7 Different flow patterns on gabion weirs; (a) Complete over-topping flow condition; (b) Seepage flow face with no-overtopping

Fig. 8 Overflow and through flow jets
Seepage flow forms, in general, when $H/L < 0.108$. Seepage profiles for flow over the gabion weir are presented in Fig. 9 for different particle sizes. As evident from the figure, decreasing the particle size leads to an increase in the approach water level and the seepage flow is transformed to partial overtopping conditions. This was caused by the reduction of through flow discharge with decreasing the particle sizes.

![Seepage flow profiles for a gabion weir with $L = 0.25$ m, $\theta = 45^\circ$](image)

Figure 9 Seepage flow profiles for a gabion weir with $L = 0.25$ m, $\theta = 45^\circ$

Figure 10 represents the flow behavior around gabion weirs with different side slopes. With increasing flow discharge, complete overflow occurred. The flow over the gabion weir’s crest was spatially varying flow with either increasing or decreasing discharge. The decreasing discharge appeared over the crest near the upstream edge once a portion of the flow entered the porous domain. Due to the flow resistance of porous media, some streamlines exited from the weir’s crest. In fact, with increasing discharge, spatially varied flow appeared on the crest. Again, in downstream portions, flow entered into the porous media as a consequence of pressure differences...
between inside and outside of the weir. In fact, the difference between pressure between the inside and outside of the permeable body can change the flow conditions.

A reduction in the slope of the upstream face leads to an increased curvature of streamlines and the velocity distribution exhibits a non-uniform wavy shape due to the geometrical characteristics of the weirs and the interaction between the through flow and the flow passing above the weir. As the velocity profiles move downstream, the velocity distribution within the porous structures were more affected by the presence of the pores. The velocity magnitudes of the fluid over and inside the porous-fluid interface differed and the crest of the weir behaves somewhat like a slip boundary condition (Fig. 11).
Fig. 10 Streamlines around and through the gabion weirs with $L = 50$ cm and GP1 filling material (a) rectangular weir, $Q = 21.4$ l/s, (b) rectangular weir, $Q = 32.4$ l/s, (c) $\theta = 45^\circ$, $Q = 21.4$ l/s, (d) $\theta = 45^\circ$, $Q = 32.4$ l/s, (e) $\theta = 26.5^\circ$, $Q = 21.4$ l/s, (f) $\theta = 26.5^\circ$, $Q = 32.4$ l/s

For complete overflow conditions, nappe, transition, and skimming flows were observed for the gabion weir. With overflow conditions, the gabion weirs were fully saturated. A portion of the approach flow passed over the weir and emerged as a free-falling jet. The jet of through flow from the downstream vertical edge affected the circulation cavity downstream of the weirs. Downstream of the rectangular gabion weir, a clockwise rotating flow was observed. With increasing the mean particle size, the water jets exited from higher layers at the downstream edge and the nappe thickness was relatively slender than that of a solid weir. Observations showed that rectangular gabion weirs, compared to a solid weir, lead to greater cavity depth and a local submergence zone.
appeared just downstream of the vertical edge. Figure 12 shows the effects of particle diameter on the nappe thickness and the cavity just downstream of the rectangular weir. The GP3 model resulted in a bigger cavity formed downstream the weir.

![Image of jet through flow and the circulation cavity downstream of the rectangular gabion weirs for Q = 21.4 (l/s), (a) GP2, (b) GP3](image.png)

Fig. 12 Jet through flow and the circulation cavity downstream of the rectangular gabion weirs for \( Q = 21.4 \) (l/s), (a) GP2, (b) GP3

Figure 13 presents the effects of different average particle diameters and side slopes (\( \theta \)) on the water surface profile for gabion weirs with \( L = 0.25 \) m. It is observed that the approach flow head increased as particle sizes decreased – a consequence of increasing flow resistance. Moreover, results indicate that the upstream water level over the sloped gabion weirs was greater than that observed over the gabion rectangular weir. For constant crest length and flow discharge, the upstream water level over the gabion weir with \( \theta = 26.5^\circ \) exceeds that for the weir with \( \theta = 45^\circ \).
4.3. Discharge coefficients

The free-flow discharge coefficients were computed for all weirs and results are shown in Fig. 14. The porosity of the permeable weirs induced some flow through the weir’s body. Therefore, in general, gabion weirs have greater discharge coefficients of solid weirs. In Fig. 14(a), all the results fell within a narrow crest weir range \((0.35 < H/L < 1.5)\); however, in Fig. 14(b), most of results were in the range \((0.1 < H/L < 0.35)\). For the same inlet discharge, doubling the crest’s length led to
to a decrease in the discharge coefficient. Since the crest length over gabion weirs became larger ($L = 0.25$ and $0.5$ m), there was a decreasing value of average discharge coefficients to $1.24$ and $1.04$, respectively. As a result, the discharge coefficients for a broad-crested weir were less than those for a narrow weir due to the increased porous media and resistance against the flow.

Comparing the discharge coefficients of rectangular and embankment weirs indicated that the discharge coefficient decreased as the upstream face slope changed from $90^\circ$ to $26.5^\circ$. For narrow gabion weirs, as $\theta$ changed from $90^\circ$ to $45^\circ$ and then to $26.5^\circ$, the average $C_d$ value became $1.33$, $1.22$, and $1.17$, respectively. However, the corresponding average $C_d$ value for gabion broad-crested weirs was $1.07$, $1.05$, and $1.01$. In other words, the largest average discharge coefficient occurred when the rectangular gabion weir was filled by GP1 whereas for a weir with $\theta = 26.5^\circ$ the GP3 material resulted in the smallest discharge coefficient.

Increasing the porosity or average particle size led to a higher discharge coefficient for the narrow embankment weirs compared to the broad-crested weirs. For all gabion weirs, the larger the average particles size, the higher discharge coefficient.

Unlike the solid weir, the discharge coefficient had a strictly decreasing trend in relation with $H/L$. As $H/L$ increased, the experimental data converged to the results of a solid weir. Indeed, the discharge coefficient values for broad-crested solid and gabion weirs with different porosities approached constant values for $H/L < 0.3$. For narrow weirs, $C_d$ reached values similar to those of the solid weir for $H/L > 0.8$ which agrees with what was reported by Safarzadeh and Mohajeri (2018) and Salmasi et al. (2020).

Since the accurate direct measurement of through flow and overflow in complex hydrodynamic conditions are often not possible, numerical simulations were used to estimate through flow and
overflow. Figure 15 shows the effect of different slopes of embankment weirs on the ratio of inflow to total discharge \(Q_{in}/Q\) and the ratio of overflow discharge to total discharge \(Q_{over}/Q\). Changing the side slopes from 26.5° to 90° reduced the overflow discharge above the crest of weir and the portion of the approach flow that passed through the weir body increased. For instance, for gabion weirs with \(L = 0.5\) m and with GP1, the average \(Q_{in}/Q\) ratio of weirs with side slopes of 90° and 45° were 42.6% and 25.2%, respectively. These values are larger than that for gabion weirs with a side slope 26.5°. Indeed, the through flow portion was reduced because of the increasing pore length inside the permeable body and increasing resistance of the porous medium against the approach flow as suggested by the Darcy-Weisbach equation.

The results indicate that overflow discharge increases with increasing \(H/L\) ratio. The primary reason for decreasing the through flow was the reduction of flow passing through the submerged permeable body, which allowed a larger portion of flow to pass over the weir. With changing \(H/L\) from 0.20 to 0.43, the \(Q_{in}/Q\) ratio of broad-crested weirs with slopes of 90°, 45° and 26.5° decreased 61%, 55%, and 53%, respectively. These results revealed that the cavity downstream of rectangular gabion weirs intensified the effect of submergence on through flow discharge.

![Discharge coefficients (C_d) variation with H/L, (a) L = 0.25, (b) L = 0.5 m](image-url)
4.4. Energy dissipation

In the present study, Bernoulli’s equation was used to calculate the upstream and downstream total energy for flow over gabion weirs. Both upstream and downstream water depths were measured in the center of the flume. The tail water depth was measured where there were few bubbles and less undulation in the tail water. Upstream ($E_u$) and downstream ($E_d$) total energy and relative energy dissipation ($\Delta E$) were calculated as follow:

$$E_u = P + H + \frac{v_u^2}{2g}$$  \hspace{1cm} (18)

$$E_d = y_2 + \frac{v_d^2}{2g}$$  \hspace{1cm} (19)

$$\frac{\Delta E}{E_u} = \frac{E_u - E_d}{E_u}$$  \hspace{1cm} (20)
where, \( P \) is the weir height, \( H \) is the upstream water head over the crest, \( y_2 \) is the tail water depth, \( V_u \) and \( V_d \) are upstream and downstream mean velocities, respectively. The velocities were calculated using the continuity equation.

A hydraulic jump is a common phenomenon in the hydraulics of rectangular weirs that increases the energy dissipation. In order to eliminate the effects of hydraulic jump on relative energy dissipation, the initial depth upstream of the hydraulic jump was used to estimate the downstream total energy. Bélanger’s equation was used to calculate the conjugate water depths for the hydraulic jump; the equation is expressed by:

\[
\frac{y_2}{y_1} = \frac{1}{2} \times \left( \sqrt{1 + 8 F_{r1}^2} - 1 \right)
\]

(21)

Subscripts 1 and 2 refer to upstream and downstream flow locations, respectively. \( F_{r1} \) is upstream Froude number and is equal to \( \sqrt{\frac{v_1}{g(y_1 + P)}} \) in a rectangular canal.

Figure 16 shows the effects of particle size and geometric configuration on relative energy dissipation. As the discharge increases or as the \( H/L \) ratio increases, relative energy dissipation is reduced up to 45% with gabion weirs. This reduction is due to the reduction of the effects of porous media on the approach flow. In Fig. 16(a), a comparison between the energy dissipation of rectangular and embankment weirs showed larger \( \Delta E/E_u \) for embankment weirs. Increasing the upstream and downstream slopes increased the resistance and energy dissipation because of the longer length of the permeable region. A comparison between different particle sizes indicates that decreasing the particle average size leads to greater energy dissipation.

Total hydraulic head contours along the channel are shown in Fig. 17. For the same porous properties, changing the side slopes from 90° to 26.5° decreases the total hydraulic head over and
inside the gabion weir. As shown, most energy dissipation occurs in the rectangular and
downstream sections of the embankment weirs.

Fig. 16 Energy dissipation vs. $H/L$, (a) narrow weirs, (b) broad-crested weirs

Fig. 17 Total hydraulic head contours for weirs with $L = 0.5$ m for $Q = 21.4$ (l/s) and GP1, (a) rectangular
weir, (b) weir with $\theta = 45^\circ$, (c) weir with $\theta = 26.5^\circ$
Conclusions

Based on results set forth in this study, the following conclusions can be drawn.

- Hydrodynamics behind the flow at the porous-fluid interface is complicated because of the inherent complexity of the through flow and its interaction with the flow that passes over the crest of weir. Such effects make the weir more efficient.
- For gabion weirs, seepage flow and complete overflow was observed. The seepage flow can be classified as either through flow with no-overtopping or through flow with partial overtopping.
- Three-dimensional simulations were developed and calibrated against experimental data. In the calibration step, a decrease in mean grain diameter and porosity results in an increase in the $\alpha^*$ value and the $B$ coefficient.
- Unlike the embankment solid weirs, the discharge coefficient tends to decrease with decreasing the side ramp slopes. Also, with increasing $H/L$, the discharge coefficients of gabion weirs decreases.
- Decreasing the upstream face slope makes the approaching streamlines exhibit strong curvature and become more oriented. The portion of the approach flow passing over the weir crest increases and the head upstream the weir increases significantly.
- At low discharges, a strong relative energy dissipation ($\sim 70\%$) was observed. However, with increasing the $H/L$ ratio, relative energy dissipation was reduced up to $45\%$ in gabion weirs.
Notation

\( a \) = Resistance coefficient (s/m);

\( A \) = Experimental coefficient;

\( b \) = Resistance coefficient (s\(^2\)/m\(^2\));

\( B \) = Experimental coefficient;

\( C_d \) = Discharge coefficient;

\( d \) = Drag term in Navier-Stokes equation;

\( d_x \) = Drag term in x direction in Navier-Stokes equation;

\( d_y \) = Drag term in y direction in Navier-Stokes equation;

\( d_m \) = Average diameter of the particles (m);

\( d_z \) = Drag term in z direction in Navier-Stokes equation;

\( E_u \) = Upstream energy (m);

\( E_d \) = Downstream energy (m);

\( f_b \) = Body forces (N/m\(^2\));

\( f_1 \) = Functional symbol;

\( f_2 \) = Functional symbol;

\( f_3 \) = Functional symbol;

\( f_x \) = Body force in x direction (N/m\(^2\));
$f_y = \text{Body force in y direction (N/m}^2\);$

$f_z = \text{Body force in z direction (N/m}^2\);$

$F_d = \text{Drag coefficient;}

Fr = \text{Froude number;}

$g = \text{Gravitational acceleration (m/s}^2\);$

$H = \text{Upstream water head over the crest (m);}$

$H_t = \text{Upstream total head (m);}$

$i = \text{Hydraulic gradient;}

L = \text{Length of crest (m);}$

$M = \text{Normal distance from the solid surface of weir (m);}$

$n = \text{Porosity (%);}$

$p = \text{Pressure (Pa);}$

$P = \text{Height of weir (m);}$

$Q = \text{Inlet flow discharge (m}^3/s\);$

$Q_{in} = \text{Discharge passing through the permeable body of weirs (m}^3/s\);$

$Q_{over} = \text{Overflow discharge (m}^3/s\);$

$t = \text{Time (s);}$

$u = \text{Velocities in x direction (m/s);}
\( u^* \) = Shear velocity (m/s);

\( U \) = Velocity (m/s);

\( V \) = Velocity in y direction (m/s);

\( V_u \) = Upstream velocity (m/s);

\( V_d \) = Downstream velocity (m/s);

\( w \) = Velocities in z direction (m/s)

\( W \) = Width of weir (m);

\( y_1 \) and \( y_2 \) = Conjugate water depths of hydraulic jump (m);

\( Y^+ \) = Cell size near to solid boundary;

\( \alpha \) = Numerator of the B coefficient;

\( \alpha^* \) = Suitable numerator of the B coefficient;

\( \theta \) = Upstream and downstream slope of the weir (Degree);

\( \nabla \) = Gradient operation;

\( \mu \) = Dynamic water viscosity (Pa · s);

\( \rho \) = Water density (kg/m\(^3\));

\( T \) = Shear stress (N/m\(^2\)); and

\( \varphi \) = Fluid kinematic viscosity (m\(^2\)/s).
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Figure 1
Scheme of water supply system

Figure 2
Tested solid and gabion weirs
Figure 3

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Figure 4

Granulation curves for three different particle diameter populations
Figure 5

2D depiction of the solution domain and boundary conditions.

Figure 6

Numerical simulation, $\alpha=1.0$
Numerical simulation, $\alpha=1.2$
Numerical simulation, $\alpha=1.8$
Numerical simulation, $\alpha=3.5$
Experimental data
Variation of predicted water surface profiles for different $\alpha$ values over a gabion broad-crested weir with $L = 0.5$ m, GP1 and $Q = 21.4$ l/s

**Figure 7**

Different flow patterns on gabion weirs; (a) Complete over-topping flow condition; (b) Seepage flow face with no-overtopping

**Figure 8**

Overflow and through flow jets
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Mean streamwise velocity profiles above and within the porous structures

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Figure 15

Calculated through flow and overflow variation with H/L for weirs with L = 0.5 and GP1, (a) Qin/Q, (b) Qover/Q
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Energy dissipation vs. H/L, (a) narrow weirs, (b) broad-crested weirs

Figure 17
Total hydraulic head contours for weirs with $L = 0.5$ m for $Q = 21.4$ (l/s) and GP1, (a) rectangular weir, (b) weir with $\theta = 45^\circ$, (c) weir with $\theta = 26.5^\circ$. 