DISCUSSIONS AND CLOSURES

Discussion of “Experimental Study of the Flow Field over Bottom Intake Racks” by Maurizio Righetti and Stefano Lanzoni


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The authors considered in their experimental study bottom racks as the typical Alpine intake structure, and proposed novel design relations based on hydraulic experimentation. In their introduction, it is stated, “The present contribution aims at investigating experimentally the validity of the simplifying assumptions ... at identifying the relevant dimensionless parameters, and at developing a physically based relationship allowing the correct design of bottom racks.” It may be considered a pity that no comparison with previous results was made, because the impact of the results would have been much larger then.

The width of the test channel was relatively small with only 0.25 m, resulting in an extremely small intake structure. Top-rounded bars were only tested, and the effect of the bar shape on the intake features was not further investigated. Are these bars really the current design basis? Or were these taken for experimental reasons? It is well known from the outflow characteristics of a tank that the authors’ outflow geometry would result in scale effects mainly due to fluid viscosity, given the extremely narrow bar openings of only 5 mm. Did the authors consider this limitation in their setup? It would be interesting to see their measurements under “static conditions,” as indicated at the bottom of page 19.

The range of the approach flow Froude number is limited to within 1.2 < Fr < 2.05: Is this really the range encountered in hydraulic practice? What would happen if subcritical approach flow occurred? Another detail of considerable importance is the exact finish of the bottom rack end, such as shown, for example, in Fig. 6. If the concrete structure is made as shown in this plot, the down-flow would be considerably deflected by the impact onto the bar anchoring and lead to other results in terms of outflow as compared to an outflow not influenced by the concrete structure. Can the authors comment this effect? A result of this finish is the dramatic increase of sin α in Fig. 7. It can easily be realized that the outflow behavior from a bottom rack therefore is divided into the uninfluenced upstream portion with a continuous decrease of sin α(x), and into the influenced downstream portion with a relatively sharp increase of sin α. The lengths of these two reaches depend obviously on the entire opening length and the exact end geometry. Did the authors account for this end-effect in terms of the outflow features?

The factor εr=0.80 as obtained from Fig. 5 was derived from one experiment. What resulted for the other tests? This may be important, given the linear relationship with the outflow. Another shortcoming of the analysis is the neglect of the bottom slope. Previous observations indeed indicated that Eq. (6) applies within extremely small bottom slopes. However, a common practice with bottom racks is to involve a considerable slope of the order of 30%, such that stones or debris is washed down from the incline and can be periodically removed. It seems that the authors varied only the approach flow depth during their tests (i.e., Fr varied exclusively with a corresponding slope adaptation). This is another important limitation, because Eq. (7) may be considered correct essentially for flows in which the bottom slope is compensated for by the friction slope. Further, the result as presented in Eq. (9) is known for decades under these particular flow conditions.

The “engineering result” of this work appears to be either Eq. (10) or (11). The streamwise coordinate x is obviously scaled with HzF10, such that it would have been normal to show this relation in Fig. 7, instead of sin α(x). Further, the authors needed three fitting parameters to express their data. Is this really required? Consider for this purpose the limiting cases (1) Fr=0 and (2) Fr→∞, for which F10=0 and F10=2. In case (1) one then has from (10) Cg=C0/0.95C00, tanh(1.852)=0.95C00, whereas C0=0 in case (2). The latter value is correct but forced to zero by the product with infinity. Is this really a robust approach, as stated in the conclusions to the paper? Can the authors comment on the physical background of this result? It is also unclear why the “static condition” was reduced by 5%.

The authors compared the results of their research with Nose-da’s results, indicating an accuracy of some ±15%. A comparison of the observed free surface profiles D(x) with computational results would be opportune because these are more sensitive to small inaccuracies in the discharge coefficient than the discharge across the bottom rack. Can the authors show free surface profiles based on their discharge equation? It is also not clear how large the minimum flow depth should be, given the authors’ statement on page 20 that problems occurred for depths less than 40 mm. It is indeed a pity that the authors’ “novel approach” remained unchecked with other data, and that no limitations of their work is indicated. These could also be stated in the closure to this discussion.

Closure to “Experimental Study of the Flow Field over Bottom Intake Racks” by Maurizio Righetti and Stefano Lanzoni


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We thank very much the discussers for his comments, which allow us to provide some clarification on our work. Before specifically addressing the various comments, it is worthwhile to briefly sum-


marize the aim of our work and the main results.

We were stimulated by the experimental results of Noseda (1955, 1956a) which, to our knowledge, provide the only estimate of the variations the discharge coefficient along a bottom rack. These variations are related to the longitudinal changes of the velocity vector just below the slit between two adjacent racks.

The energy balance along a given streamline shows that, in the absence of any approximation, the general relationship giving the discharge diverted by the rack, per unit width and unit length, reads

\[
\frac{dq}{dx} = \omega \sin \alpha \sqrt{1 + \frac{\Delta z - \Delta E}{H_0}} \sqrt{2gH_0} = \omega C_q \sqrt{2gH_0}
\] (1)

Under the assumption that energy losses tend to be balanced by the component of the weight parallel to the flow (i.e., that friction slope is roughly compensated by rack slope), the specific flow head remains constant along the rack and Eq. (1) reduces to Eq. (8) of the paper, implying that \( C_q \equiv \sin(\alpha) \). Observing that \( \sin(\alpha) \) tends to decreases almost linearly along the rack except toward its end, and stipulating that, for a vanishing velocity of the approaching flow, \( C_q \) must tend to the constant value \( C_{q0} \) measured under static conditions (i.e., in a constant head tank), resulted in Eq. (10) of the paper. Here we note that this latter equation is affected by a misprint, its actual form reading

\[
\frac{dq}{dx} = \omega \sin \alpha \sqrt{1 + \frac{\Delta z - \Delta E}{H_0}} \sqrt{2gH_0} = \omega C_q \sqrt{2gH_0}
\]
First of all, it not only accounts for the variations in 
flow along the rack. 

by the grid can be readily estimated from quantities involving

\[ C_q = C_{q0} \left( \frac{a}{H_0} F_{\text{fr}} + 1 \right) \left\{ \tanh [b_0 (\sqrt{2 - F_{\text{fr}}})] \right\}^{b_1} \] (2)

Eq. (2) has some important features that merit pointing out. First of all, it not only accounts for the variations in \( C_q \) along the rack but also relates \( C_q \) to the Froude number \( F_{\text{fr}} \) of the incoming flow (through \( F_{\text{fr}} \)), an aspect of the problem that has not been addressed so far. In other words, Eq. (2) accounts for the observational evidence that, all the other parameters remaining unchanged, the diverted discharge tends to decrease for increasing \( F_{\text{fr}} \). Second, the above equations apply also to the case of subcritical approaching flows, even though our experiments concentrated on the case of supercritical flows, a condition which often occurs in mountain streams (e.g., Bouvard 1992). Clearly, when considering subcritical flows the values of the parameters \( a, b_0 \) and \( b_1 \) could change. Nevertheless, as we will see later on, Eq. (2) leads to a reasonably good agreement with observations, independently of the supercritical or subcritical character of the flow, without resorting to any calibration of the above parameters. Finally, we observe that, as a consequence of the choice of \( H_0 \) as the relevant hydraulic head in Eq. (1), the overall discharge diverted by the grid can be readily estimated from quantities involving only the incoming flow [see Eq. (11) in the paper] and, hence, without resorting to the integration of the surface water profile along the rack.

Clearly, the reliability of the approach leading to Eq. (2) depends on the correctness of the assumption that friction slope is nearly balanced by rack slope. The experiments carried out by Venkataram (1977) (summarized in Venkataram et al. 1979) indicate that, although some reduction of the specific head along the bed rack can occur for supercritical flows, the extent of the decrease is relatively small (less than 5% according to Venkataram et al. 1979) if the rack is inclined. More recently, the experiments of Brunella et al. (2003) suggested that the specific head can be assumed approximately constant except in the portions of the rack (located toward its end) where the flow depth can become so small that friction increases significantly. Possibly, as indicated by the discussers, the assumption that \( \Delta z = \Delta H_0 \) nearly balances \( \Delta E / H_0 \) into Eq. (1) tends to fail for strongly inclined grids. In this latter case, we can assume, as a first approximation, that Eq. (2) still holds but, since the term \( \sqrt{1 + (\Delta z - \Delta E) / H_0} \) has to be retained in Eq. (1), the hydraulic head \( Y = H_0 + (\Delta z - \Delta E) \) needs to be adopted instead of \( H_0 \), thus requiring the integration of the water surface profile to estimate the energy dissipation \( \Delta E(x) \) along the rack.

Specific Points

The reasonably good results of the comparison with other independent set of data available in literature (see Fig. 8 of the paper and also the final results of this closure, which will be discussed later on) give a clear answer to some of the points raised by the discussers. Indeed, Noseda’s (1956b) experiments were carried out in a channel having a width double than that of our experiments, and using a completely different type of bars (T-shaped barcrested bars, yielding a sharp detachment of the flow) with a spacing 5.7–11.7 mm. The choice of top-rounded bars in our experiment, rather than for design purposes (these bars are not adopted in the practice), was related to the need of preventing, as much as possible, flow separation in the slot between two adjacent bars, thus avoiding multiple refractions due to vein oscillations, a fundamental requirement for reliable LDA measurements. In any case, the role of the bar shape, as well as the viscous effects possibly enhanced by the relatively small bar openings are accounted for in Eq. (2) through the discharge coefficient measured under static conditions, \( C_{q0} \). Fig. 1 of this closure reports the different behavior experienced by \( C_{q0} \) in the various experimental settings.

The successful comparison with Noseda (1956b) also suggests that Eq. (2) can be applied to subcritical approaching flows as well as to inclined grid (the grid slope ranging from 0 to 20% in Noseda (1956b) tests). To extend this conclusion to the case of slopes higher than 20%, some caution is needed. It would be surely interesting to consider the data collected by Brunella et al. (2003), which cover a wide range of rack slopes. However, these results are essentially presented in graphical form, using dimensionless quantities and, therefore, it is not possible to extract the information required for the comparison.

Another point raised by the discussers concerns the characteristics of the flow field toward the end of the rack. The grid used in our experiments leans on a cantilever and, hence, does not particularly influence the outflow below the rack (see Fig. 2 of this closure). The final effects of the stagnation point, which forms at the end of the grid when not all the discharge is diverted, owing to the supercritical character of the flow, remains localized. The coefficient \( a \) has therefore been evaluated neglecting the local increase in \( \sin(\alpha) \) at the end of the grid, thus accepting a conservative estimate of the diverted discharge.

Let us now address the remarks made by the discussers on the chosen functional form relating \( \sin(\alpha) \) to the dimensionless parameters \( x / H_0 \) and \( F_{\text{fr}} \). Fig. 3 of this closure shows the linear dependence of \( \sin(\alpha) \) from the longitudinal position along the grid, already emerging from Fig. 7 in the paper. It also indicates that the intersection of the various lines with the vertical axis increases with the modified Froude number. To account for this dependency, we introduced the hyperbolic tangent in Eq. (2). This function, in fact, tends to zero as \( F_{\text{fr}} \) approaches \( 2^{1/2} \) (the rate of decreasing depending on the exponent \( b_1 \)), while \( C_{q0} \) tends to be recovered in the limit of vanishing \( F_{\text{fr}} \) provided that a high enough value of \( b_0 \) is assumed.

Other functional relationships satisfying the two requirements listed above can be considered. For example, to identically recover \( C_{q0} \) as \( F_{\text{fr}} \) vanishes, the following two-parameter relationship can be used.
Fig. 4 of this closure shows the comparison between measured and computed data obtained by using Eq. (3), in which the coefficient $b_1$ arising from minimizing the differences between the diverted and measured diverted discharges is $b_1 = 0.478$ (instead of 0.609). Fig. 4 contains also the data derived from the experiments of Orth et al. (1954), carried out with grid inclination 10–20%, ovoidal rack profiles, and characterized by the diversion of the whole discharge. For these latter experiments, $C_q = 0.8$ has been assumed to account for the higher grid efficiency ensured by these rounded bar shapes compared to the tapered bars adopted by Noseda (1956b) having $C_q = 0.72$. The overall agreement is still reasonably good.

Following the suggestion of the discussers we have also made some more tests, reported in Figs. 5 and 6 of this closure, giving further evidence of the robustness of the proposed approach. The first test, shown in Fig. 5, concerns the comparison between computed and measured water surface profiles. As already noted by Noseda (1956b), Brunella et al., (2003) and Righetti et al. (2000), in the upstream portion of the rack, the nonhydrostatic pressure distribution induced by a significant curvature of the streamlines implies that observational points tend to lie above the computed profile. A progressively better agreement is obtained proceeding toward the end on the rack (obviously neglecting the localized effect of the stagnation point that forms when not all the discharge is diverted by the grid).

The second test is related to the length of the grid $L$, scaled by $H_0$ required to divert the entire discharge. Observing that

$$\Delta Q = 
\frac{WF_{H_0}}{H_0} \sqrt{\frac{gH_0}{2}} \left(1 - \frac{1}{c_4^2} \right)$$

from Eq. (11) of the paper we obtain

$$L = \frac{1}{2c_1} \mathrm{e} \left(-1 + \sqrt{1 - 4c_4c_1} \right),$$

where

$$c_0 = \frac{F_{H_0}^2 \left(1 - F_{H_0}^2 / 2 \right)}{\sqrt{2 \times C_q \left(\tan \left(\beta - \frac{\beta}{2} - \frac{\beta}{2} \right) \right)}}, \quad c_1 = \frac{a}{2} F_{H_0}$$

Fig. 6 of this closure shows that the agreement between observed and computed grid length is reasonably good. Interestingly, the values of $L/H_0$ estimated through Eq. (6) are generally smaller than those predicted by the relationship proposed by Brunella et al. (2003), $C_q = 0.8$ ($L/H_0 = 0.83$). This latter relationship, in fact, yields $L/H_0 = 7.2, 4.1, 3.32$ for the three conditions examined in Fig. 6.

Finally, let us clarify the typical behavior of the transversal dimensionless profile measured in a slit of the grid exemplified by Fig. 5 of the paper. To account for the nonuniformity of this velocity profile we introduced a coefficient $\alpha_{\omega}$ defined as the ratio of the mean to the maximum velocity within the slit. The value 0.8 reported for $\alpha_{\omega}$ however, resulted from a rough estimate, that is assuming a piecewise interpolation of the measured points and a linear variation between the point closer to the wall and the value zero at the wall itself. However, if a power law with exponent in the range 1/7–1/8 (typical of a smooth wall) is used to interpolate the experimental data in the wall region, $\alpha_{\omega}$ turns out to fall in the range 0.935–0.955.

Conclusions

On the basis of the issues raised by the discussers, we have provided a hopefully more conclusive clarification on the robustness of the approach proposed to estimate the overall discharge diverted by a bottom rack. The comparison with independent sets of data suggests that Eq. (2) has a range of applicability much wider than that investigated in our experiments. Indeed, the differences between computed and predicted diverted discharges are generally lower than 15% despite the fact that:

1. Ranges of the relevant flow parameters are much larger than those investigated in our experiments;
2. Rack bars are completely different shapes;
3. Nonnegligible rack inclination (up to 20%) with respect to the channel bed of a number of configurations; and
4. Subcritical or supercritical character of the approaching flow.

It is emphasized that no calibration has been performed in the comparison. The only parameter that needs to be assigned is the value of the discharge coefficient measured under static conditions, $C_q$.

Acknowledgments

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References


The authors have tackled the problem of evaluating periodic oscillation phenomenon caused by a flow over a vertical drop pool. The authors’ finding could be applied in practical designs of ladder-type (pool and weir type) fishways to avoid the occurrence of undesired oscillation for fishes in fishway pools. A mistake was found in Fig. 11, in which the region of possible occurrence of periodic oscillation was presented. From Table 1, the ranges of the critical velocity and the pool length are $U_c = 0.345 \text{ m/s}$ to $0.7155 \text{ m/s}$ and $L = 0.08 \text{ m}$ to $0.28 \text{ m}$, corresponding to an actual range of the Froude number from 0.42 to 0.56. This conflicts with the result of Fig. 11. The authors’ conclusion related to Fig. 11 does not seem to be supported by quantitative scrutiny of the data. The discusser reproduced a new Fig. 1, by using the authors’ data in Table 1. Four fishways data in Japan were also included in Fig. 1. In these fishways, periodic oscillation phenomenon and the so-called seiche phenomenon were observed. A typical example of seiche in Tenryu fishway, Yoshida River, Japan, is shown in Fig. 2. According to a definition sketch shown in Fig. 3, dimensions of the fishway partition wall and flow conditions in four fishways were summarized in Table 1 of this discussion.

**Table 1. Dimensions and Flow Conditions of Four Fishways in Seiche Phenomenon**

<table>
<thead>
<tr>
<th>Fishway name</th>
<th>Utsushi</th>
<th>Mihara</th>
<th>Tenryu</th>
<th>Miyanaka</th>
</tr>
</thead>
<tbody>
<tr>
<td>B (m)</td>
<td>0.80</td>
<td>1.50</td>
<td>1.50</td>
<td>10.00</td>
</tr>
<tr>
<td>L (m)</td>
<td>1.25</td>
<td>2.00</td>
<td>2.00</td>
<td>3.15</td>
</tr>
<tr>
<td>$b_1$ (m)</td>
<td>0.65</td>
<td>1.00</td>
<td>1.00</td>
<td>6.70</td>
</tr>
<tr>
<td>$b_2$ (m)</td>
<td>0.15</td>
<td>0.50</td>
<td>0.50</td>
<td>3.30</td>
</tr>
<tr>
<td>$H_1$ (m)</td>
<td>0.80</td>
<td>0.60</td>
<td>0.60</td>
<td>1.50</td>
</tr>
<tr>
<td>$H_2$ (m)</td>
<td>0.70</td>
<td>0.50</td>
<td>0.50</td>
<td>1.35</td>
</tr>
<tr>
<td>$y_k$ (m)</td>
<td>0.10</td>
<td>0.10</td>
<td>0.10</td>
<td>0.15</td>
</tr>
<tr>
<td>$a$ (m²)</td>
<td>0.007854</td>
<td>0.03142</td>
<td>0.03142</td>
<td>0.03142</td>
</tr>
<tr>
<td>Slope, i</td>
<td>1/12.5</td>
<td>1/10</td>
<td>1/10</td>
<td>1/15</td>
</tr>
<tr>
<td>$\Delta h$ (m)</td>
<td>0.12</td>
<td>0.20</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>$y_1$ (m)</td>
<td>0.05</td>
<td>0.10</td>
<td>0.12</td>
<td>0.18</td>
</tr>
<tr>
<td>$b_2/b_1$</td>
<td>0.231</td>
<td>0.500</td>
<td>0.500</td>
<td>0.493</td>
</tr>
<tr>
<td>$Q_2$ (m³/s)</td>
<td>0.040</td>
<td>0.193</td>
<td>0.225</td>
<td>2.610</td>
</tr>
<tr>
<td>$Q_2/Q_1$</td>
<td>1.199</td>
<td>1.414</td>
<td>1.241</td>
<td>1.223</td>
</tr>
</tbody>
</table>

Note: Subscripts 1 and 2 indicate the main portion of partition-wall, and the square-type notch portion, respectively. B = width of fishway (m); $B = b_1 + b_2$; $Q$ = Discharge (m³/sec); $Q_2$ = total discharge in the fishway (m³/sec); $b$ = width of partition-wall (m); $y$ = flow depth (m); $y_k$ = height of notch (m); $a = \text{area of submerged orifice (m}^2\text{)}$; $\Delta h = \text{height difference (m) between the upstream and downstream partition walls}$; $y_2 = y_1 + y_k$ (flow depth in the square-type notch (m)); $H_1 = \text{height of partition wall (m)}$; $H_2 = \text{height of partition wall in the square-type notch (m)}$; and $L = \text{length of fishway pool (m)}$. 

Fig. 1. Region of possible occurrence of periodic oscillation

Fig. 2. Typical example of seiche in Tenryu fishway (Yoshida River, Gifu, Japan)

Fig. 3. Definition sketch of ladder type fishway
Almost all fishways are usually designed in the range of 0.05 < Δh/L < 0.2. The most commonly used value of Δh/L in Japan is 0.1.

Authors’ Erratum

The last line (Series U) of column Uc (cm/s) in Table 1 on page 952 should be replaced with 59.19 instead of 89.19

Closure to “Periodic Oscillation Caused by a Flow over a Vertical Drop Pool” by Chang Lin, Shih-Chun Hsieh, Kai-Joe Kuo, and Kuang-An Chang

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The writers thank the discusser for his interest in the paper and his careful inspection and correction of the mistake in Fig. 11 in the original publication. Fig. 11 in the original publication was mistakenly plotted based on the experimental data in the paper (the data are correct) and an erratum for the figure has been submitted by the authors and accepted for publication by the Journal.

The discussion may have significantly improved the quality of the original paper. Not only does the discusser point out the mistake and correct the figure, the discusser also provided four practical cases on the occurrence of the undesired oscillation in fishway pools in Japan. In addition, through the discusser’s effort the application may have been extended to the practical design of the ladder type (pool-and-weir type) structures. Although the geometry of the fishway pools is more complicated than the ideal geometry used in the present study, a similar oscillatory phenomenon (or seiche as the discusser called it) was observed. Since the present study was conducted in three small laboratory flumes and the results have never been validated by large-scale field data, the observation from the fishway pools is very valuable to verify the existence of the phenomenon.

As the discusser pointed out, the fishway pools were designed in the range 0.05 < Δh/L < 0.2 (see Fig. 1 for definition of the variables). In the corrected Fig. 11 we do not have laboratory data in that particular range with the occurrence of flow oscillation. As a result, we added the three data points provided by the discusser to the figure and presented in Fig. 2. Again, due to a lack of laboratory data the extended dashed lines at Δh/L < 0.2 are from extrapolation based on the measurement and field data. Although a small number of laboratory data are in the range of Δh/L < 0.2, they all fall in the skimming flow regime (i.e., above the upper extrapolated dashed line).

In addition to the relatively complicated geometry of the fishway pools to the ideal geometry used in the present study, the end sill length would also have some influence to the oscillation phenomenon. The laboratory data shown in Fig. 2 were taken under the experimental condition that the end sill length, Ls, is 0.5 m, greater than 1.7 times the pool length, L. That was to avoid the effect due to the limited length of end sill on the occurrence and frequency of the oscillatory flow. However, the end sill length of
the fishway pools shown in Fig. 2 by the discusser is obviously shorter than the pool length. Based on unpublished data collected by the authors, the relation between the weighted Strouhal number $S'_t$ and the normalized end sill length $L_s/L$ is plotted in Fig. 3. It shows that $S'_t$ increases monotonically for $L_s/L < 1$. The overall trend of $S'_t$ then becomes stabilized for $L_s/L > 1$. Based on the measurement results, $L_s/L$ would still influence the occurrence and frequency of the periodic oscillation as long as $L_s$ is relatively short, even if the unit discharge $q$, $\Delta h/L$, and $U_c/gL$ were kept constant.

Discussion of “Hydraulic Characteristics of Gabion-Stepped Weirs” by Chaiyuth Chinnarasri, Somachai Donjadee, and Udomsak Israngkura

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The authors have studied the important characteristics of flow over gabion-stepped weirs by conducting a detailed model study and suggested Eq. (2) of the paper as a rough estimate of the occurrence of skimming flow on gabion-stepped weirs. The discussers developed the following equation for the onset of skimming flow on gabion-stepped weirs based on available experimental data (Kells 1995; Peyras et al. 1992; Stephenson 1979; Degoutte et al. 1992)

$$\ln \frac{d}{h} = -0.2863(h/l) + 0.863$$  \hspace{1cm} (1)

with a coefficient of determination of 0.80.

A comparison of Eq. (2) of the paper and Eq. (1) of this discussion with experimental data are shown in Fig. 1. Eq. (1) gives a better fit with an average error of $-0.5\%$, while Eq. (2) of the paper has an average error of $-6.4\%$.

Based on experimental observations, Peyras et al. (1992) suggested the following equation for calculation of energy dissipation over gabion weir for $h/l=1$

$$\frac{1}{1-y} = 0.238D^{-0.526}$$  \hspace{1cm} (2)

where $y=E_l/H_T=$energy loss across the weir; and $D=q^2/gh^3=$drop number.

The range of $D$ for Eq. (4) of the paper suggested by the authors varies from $100*10^{-6}$ to $310*10^{-6}$, whereas, the range $D$ for Eq. (2) of this discussion and suggested by Peyras et al. is much larger, varying from $300*10^{-6}$ to $200*10^{-4}$. For $D = 300*10^{-6}$, using Eq. (4), $E_l/H_T=0.94$, using Eq. (2) of this discussion, $E_l/H_T=0.79$.

Amador et al. (2005) and Frizell and Mefford (1991) studied the pressure variation on horizontal impervious steps of stepped spillways, and observed the maximum pressure located on the outer edge of the step and minimum pressure (even negative) on the inner region of step. This is a similar observation shown by the authors in Fig. 6 of the paper.

Acknowledgments

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Fig. 1. Onset of skimming flow on a gabion-stepped weir

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The discussers attempted to develop an equation for the onset of skimming flow by using the experimental data of Kells (1995), Peyras et al. (1992), Stephenson (1979), and Degouette et al. (1992). However, these available experimental data were obtained from models with few steps. When the maximum unit discharge and the step height of the mentioned data were substituted in Eq. (2), the number of steps that allows uniform flow over the stepped chutes is always greater than six. Therefore Eq. (1) of the discussion is not applicable to the case of the onset of skimming flow under uniform flow conditions.

In addition, the existence of an infiltration discharge decreases the critical flow depth ($d_c$) over stepped spillways (Degouette et al. 1992). In the experiments of Peyras et al. (1992), the amount of infiltration discharge through the gabion showed an effect on the onset of skimming flow. For a high base flow, the value of $d_c/h$ is therefore lower than that for low base flow. That is why Eq. (1) of the discussion shows a slightly lower value of $d_c/h$ for the onset of skimming flow than Eq. (2) of the technical note.

Energy Dissipation of Gabion-Stepped Weirs

The discussers referred to an equation of Peyras et al. (1992) for the calculation of energy dissipation over a gabion weir for $h/l = 1$ as given in Eq. (2) of the discussion. However, it must be emphasized that in the experiments of Peyras et al. (1992), each gabion was placed on the top of another gabion to form the downstream side of the weir as shown in Fig. 1(a). The models had downstream slopes of 1/1, 1/2, and 1/3 with three, four, or five steps. A large portion of the base flow can flow through the gabion from top to bottom. In this case, the base flow portion is much greater than that of the present study in which each gabion was placed on the impervious step as shown in Fig. 1(b). The energy dissipation of flow over the gabion placed on an impervious step is different from that of flow over a gabion placed on top of another gabion. Generally, for the same downstream slope, much of the energy of the flow over a gabion placed on an impervious step is dissipated due to the recirculating vortices between the steps and the form-drag effects of the step shape. A little of the energy dissipation is due to flow through gabion voids.

It can be concluded that the reasons for differences in energy dissipation are the design of gabion steps and the numbers of steps. The first factor is related to the amount of base flow while the second factor is related to the uniform flow condition. Minor differences arise from the type of entrance to the stepped channels, gabion sizes, stone porosity, measurements of the residual flow energy, slope of stepped channels, and effects of turbulence. For various stepped chutes, the factors influencing energy loss were described by Chinnarasri and Wongwises (2005).
References


