Discussion of "Computing Nonhydrostatic Shallow-Water Flow over Steep Terrain" by Roger P. Denlinger and Daniel R. H. O'Connell

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The authors are to be commended for their valuable and unique comparison of experimental data for clear-water dam-break floods on steep slopes and for their nonhydrostatic shallow-water model. The dam-break experiments down a steep slope at the USGS outdoor laboratory (Logan and Iverson 2007) mimic the behavior of natural flows and shed light on different mechanisms (inertia, viscous dissipation, pressure gradient) that control the flow dynamics. The discusser has studied roll-wave development in dam breaks and similar problems on steep slopes restricted to relatively long timescales, for which the kinematic regime is reached, and the authors' fine results present a great opportunity to consider some interesting features of the dam-break wave.

It is understood that there is a lack of validation of asymptotic solutions for dam-break waves on steep slopes based on the kinematic wave approximation (Lighthill and Whitham 1955) with both experimental data and numerical simulations. Even the earlier solutions by Hunt (1982, 1984) and Weir (1983) for shallow slopes have not yet been validated. As discussed by Hunt (1984): "A comparison with experiment shows good qualitative agreement, but more comparisons with experiment should be made in order to assess the qualitative accuracy of the solution." This question remains still unresolved (e.g., Singh 2002, and references therein).

To address this point, the value of enhanced gravity g' must first be estimated. For a uniformly sloping bed inclined at an angle θ with respect to the horizontal, mathematical similarity between authors' equations and those proposed by Dressler (1978) implies $g'=g\cos^2\theta$. (To deduce this value, the reader must take into account that the streamwise coordinate in Dressler's equations is oriented with the bottom bed.) Indeed, dam-break solutions on steep slopes using one-dimensional shallow-water modeling with $g'\equiv g\cos^2\theta$ have become a popular hydraulic tool and have been used to refine estimates of former solutions for shallow slopes in a variety of settings (e.g., Ancey et al. 2008; Bohorquez and Fernandez-Feria 2008). How precise is this estimation with respect to that computed by the authors? Supposing this value of reduced gravity, the one-dimensional differential version of Eqs. (13–15) corresponds to Saint-Venant equations (strictly valid for shallow slopes) but rescaled as a function of constant parameters. Thus, the outer solutions for floods of point mass sources by Weir (1983) and Hunt (1984) are applicable to steep slopes after rewriting them in the appropriate set of dimensional variables

$$h(x,t)=\frac{fu^2}{8g\tan\theta},\quad u(x,t)=\frac{2x}{3t}$$
$$\text{for } 0\leqslant x\leqslant x_s(t)\equiv\left(\frac{54g\tan\theta}{f}\frac{V}{B}t^2\right)^{1/3} \tag{1}$$

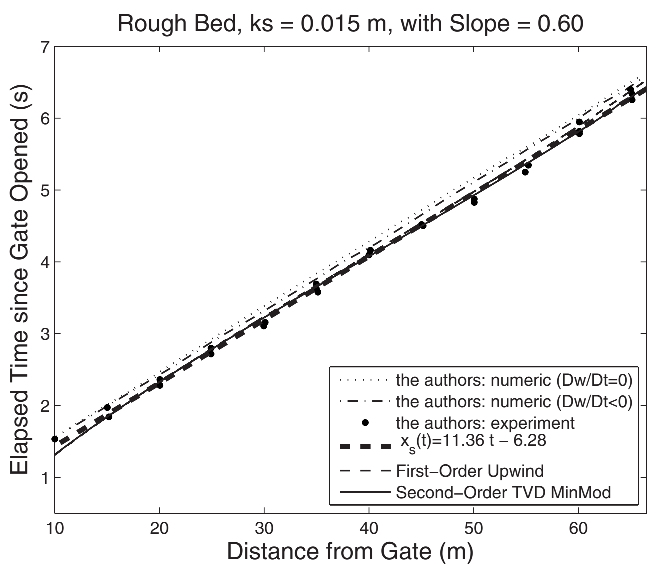

Fig. 1. Temporal evolution of the water front obtained with Darcy-Weisbach friction factor (after Bohorquez and Fernandez-Feria 2008) compared with the experimental and numerical results by the authors

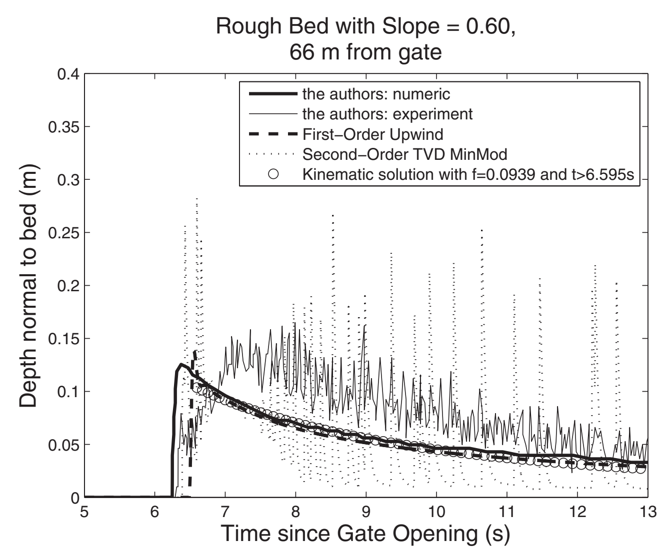

Fig. 2. Comparison of the water depth recorded 66 m from the gate with the numerical simulations of the authors and that obtained with Darcy-Weisbach friction factor (after Bohorquez and Fernandez-Feria 2008), the asymptotic solution Eq. (1) for the constant value of $f=0.0939$ is also shown

Here, f=Darcy-Weisbach friction factor; V=reservoir volume; B=channel width; and x_s=shock location.

The main drawback of Eq. (1) is that f is assumed constant, which implies that $x_s(t) \sim t^{2/3}$. However, the authors' experimental data for a rough bed, k_s=0.015 m with slope 0.60, fits the linear law $x_s(t) \approx 11.36t - 6.28$ (coefficient of determination=0.9998), as shown in Fig. 1 in this discussion. The difference in the temporal exponent of both solutions causes a large discrepancy between the predicted and the experimental values of x_s. Consequently, the full nonlinear shallow-water equations must be solved numerically in order to capture with accuracy the advancing of the wetting front. In so doing, as described in Bohorquez and Fernandez-Feria (2008) for $V=6\ \mathrm{m}^3$, the discusser finds an excellent agreement between the numerical results and those reported by the authors (see Fig. 1). In this plot, the first-order upwind numerical simulation is performed with mesh size Δx=0.05 m. The mesh size was decreased to Δx=0.003 m when employing the MinMod total variation diminishing (TVD) method, which is second-order accurate in both space and time. The mesh size was decreased to assess the effect of reducing the numerical diffusion in the simulations, as discussed below. The Courant-Friedrichs-Lewy (CFL) number is fixed to 0.045.

On the other hand, predicted values of $h(x,t)$ from Eq. (1) at late time, $t > 6.6$ s, are in better agreement with the numerical simulations. The upwind numerical simulation allows an objective quantification of the Darcy-Weisbach friction factor, the inferred mean value at x=66 m is f=0.0939, with a standard deviation of 0.0190. This finding justifies the constant value of the f parameter in the local evaluation of the flood depth h in Eq. (1). The agreement between the asymptotic and the upwind solution at x=66 m (see Fig. 2) suggests that the kinematic wave approximation is not unreasonable. Both solutions are also very close to the authors' but underpredict experimental measurements. Conversely, the experimental water depth falls within the range of the TVD numerical predictions owing the development of *natural* roll waves in the numerical model. Since the TVD method is second-order accurate in both space and time and the mesh size is much finer than that employed in the upwind numerical simulation, the numerical diffusion is negligible with respect to the upwind simulation. According to stability analyses of kinematic waves (see Lighthill and Whitham 1955; Bohorquez 2008), dynamic waves compete with kinematic waves and induce the appearance of roll waves for Froude numbers (F) larger than a critical value F_{cr}, which depends on the local slope of the free-surface height and tends to the threshold F_{cr}=2 of the uniform stream as time increases. In the kinematic regime, the Froude number is given by

$$\mathsf{F} \equiv \frac{u}{\sqrt{gh}} = \sqrt{\frac{8}{f}\tan\theta} \tag{2}$$

which is F=7.15 at x=66 m. Hence, owing to the unstable nature of the kinematic wave, one should observe the amplification of numerical noise when using a low-diffusive Riemann solver, as in the experiments conducted on August 30, 1994, August 28, 2002, and June 7, 2006, at the U.S. Geological Survey Debris-Flow Flume, where instabilities develop at a late time (Logan and Iverson 2007 includes the complete films). Did the authors find roll waves in their numerical simulations as in the physical experiments?

As the authors commented, their equations avoid the use of curvilinear coordinates by means of enhanced gravity. Consequently, the authors' contribution is welcomed for supplying a simple way to account for nonhydrostatic effects that allows for an efficient update of Riemann solvers for Saint-Venant equations to cope with such phenomena in this difficult area.

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We would like to thank the discusser for his comments and valuable additional analysis, which provide further validation for our work and also afford discussion of an additional point. In response to his question about roll waves, the answer is that we severely damped roll waves with our solution technique. Roll waves are damped in our formulation by the limiter used for getting acceleration of surface elevation to obtain average acceleration over depth. Specifically we place a limiter on $\partial h/\partial t$ in the calculation of vertical velocity and then average vertical acceleration over a 3-by-3 grid of cells for each cell evaluated. The damped waves are barely visible in the heavy black curve in Fig. 2 in the discussion, where it can be seen that they increase in wavelength from the front of the flow to the back.

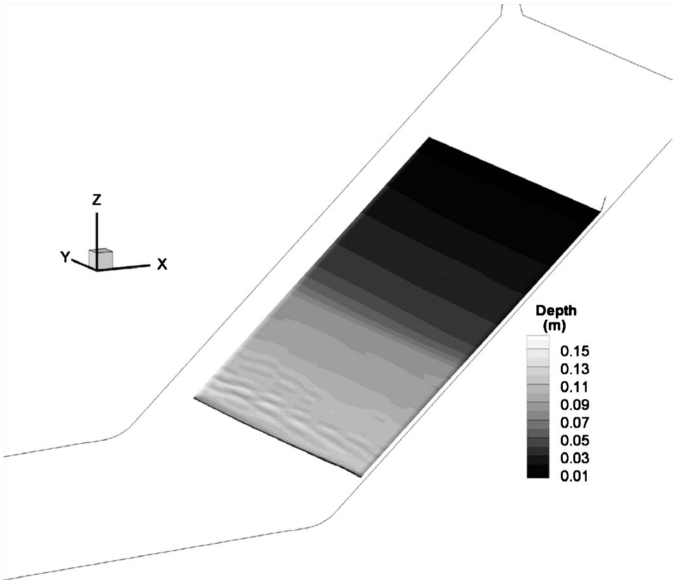

Fig. 1. Roll waves developed in simulated dam-break flow when front is 66 m from the gate in the USGS flume (Logan and Iverson 2007, courtesy of the U.S. Geological Survey); the waves diminish in amplitude from the front toward the back and are similar in amplitude and wavelength to those calculated by the discusser using a comparable finite volume method

If the limiter is shut off, then roll waves appear, as shown in Fig. 1 in this closure. This is a snapshot of the same dam-break simulation at $x=66$ m as in the discussion's Fig. 2, but without the limiter activated. Roll waves appear near the front and become damped with distance from the front. The amplitude and spacing of the waves are comparable to those calculated by the discusser near the front of the flow and are also consistent with the wave estimates of Balmforth and Mandre (2004) for the Froude number of the flow and surface roughness of the flume.

This approach, however, cannot be used in routing flows over real terrain with our existing formulation. When we remove the limiter from the vertical acceleration calculation and the topographic loading has strong local variability, we produce large "rogue" waves and carbuncle-like instabilities. That is why we use limiters for vertical acceleration when simulating dam-break scenarios such as Malpasset.

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Discussion of "Generalized Flow Rating Equations at Prototype Gated Spillways" by Matahel Ansar and Zhiming Chen

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The authors are to be congratulated for providing a detailed study of generalized flow rating equations for flow over ogee-shaped and trapezoidal-shaped spillway. According to Eq. (11), the predictions with the single generalized equation are continuous across the different flow conditions. Hence, the discussers would like to example whether the proposed single generalized equation is capable of predicting all flow conditions. In calculation for free weir flow, $y_c < H$ according to Eq. (8) when $a_4=0.71$, but $y_c > H$ according to Eq. (11) when c_1, c_2, c_3 are 0.91, 0.28, 0.30, respectively. It is doubtful that $H < y_c$ for free weir flow. In fact, Eq. (8) is correct for free weir flow, but Eq. (11) is doubtful for calculating the discharge of free weir Flow when $h < 0$.

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The authors have made a very useful contribution to discharge estimation at gated spillways and weirs. Based on an extensive field data set, they proposed a generalized flow rating equation applicable to different flow conditions, namely, submerged orifice, submerged weir, free-orifice, and free-weir flows. The application of this equation is more suited to relatively low spillways and weir heights, low headwater depths, and flows controlled by vertical gates. Nevertheless, the generalized flow equation establishes the mathematical and physical continuity between the different flow conditions, which makes the use of this equation an advantage in practical designs of gated spillways and weirs, mainly in cases of large tailwater level variance. This discussion is intended to supplement the authors' contribution by more closely examining the results obtained using the equation in different flow conditions.

Free Orifice

The single-generalized equation becomes

$$Q = A\sqrt{gH}\left(0.91 + 0.28\frac{G_0}{H}\right)^{3/2} \tag{1}$$

where A=orifice area.

If $G_0 \ll H$ (not the case considered by the authors), the discharge coefficient c_d in the equation

$$Q = c_d A\sqrt{2gH} \tag{2}$$

takes the value 0.614, which is acceptable.

A typical field situation of this study could be: L (sill length) $=1$ m; $G_0=1$ m; and $H=2$ m. In this case, Q, according to Eq. (1), takes the value 4.77 m^3/s. Introducing this value in Eq. (2), $c_d = 0.88$ is obtained. This value seems too large, even considering that lateral and floor contractions are eliminated. It should be noted that (a) even with a low ratio H/G_0, Eq. (2) could be used (Lencastre 1996), and (b) the calculation was realized with H referring to the orifice axis (H=1.5 m).

Free Weir

The single-generalized equation becomes

$$Q = c_d L\sqrt{2g}H^{3/2} \tag{3}$$

where $c_d=(0.91+0.28)^{3/2}/\sqrt{2}=0.918$, which is unacceptable. For instance, in a typical field situation of this study (ogee-shaped spillway, vertical upstream face, and sill height=design head=H), c_d, according to Lencastre (1996), takes the value 0.487.

Submergence Factor

According to the single-generalized equation, the submergence factor, in both cases, is $(1-h/H)^{0.45}$, that is

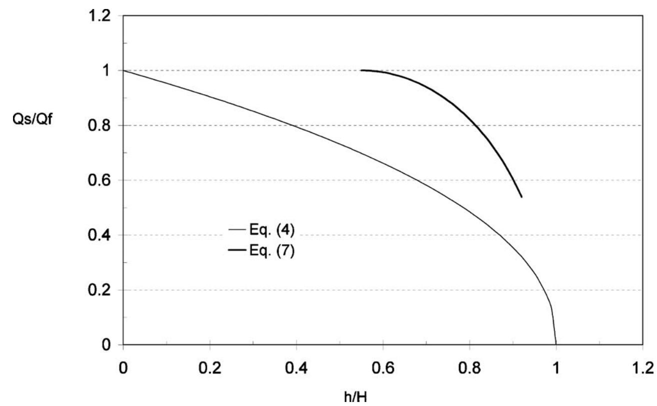

Fig. 1. Variation of Q_s/Q_f with h/H

$$\frac{Q_s}{Q_f} = \left(1 - \frac{h}{H}\right)^{0.45} \tag{4}$$

where Q_s=discharge (submerged flow condition); and Q_f =discharge (free-flow condition).

Submerged Orifice

Lencastre (1996) proposes

$$Q_s = c_{d1}A\sqrt{2g(H-h)} \tag{5}$$

for totally submerged orifices, and

$$Q_s = c_{d2}Lh\sqrt{2g(H-h)} + \frac{2}{3}c_{d3}L\sqrt{2g}[(H-h)^{3/2} - (H-G_0)^{3/2}] \tag{6}$$

for partially submerged orifices, where c_{d1}, c_{d2}, and c_{d3} are discharge coefficients. According to Lencastre (1996) in page 323, the values of these discharge coefficients are "not very well known," and it is admissible to use the corresponding values for free orifices.

Considering the example presented in the section "Free Orifice," the Table 1 was created. The values given by Eq. (4) are on the safe side (as regards upstream levels). Taking into account the fact that Eq. (4) is obtained from prototype data and the uncertainties mentioned by Lencastre, it seems reasonable to use Eq. (4) when calculating submerged orifice flow.

Submerged Weir

Based on the results presented by Tullis and Neilson (2008), an alternative estimate of Q_s/Q_f can be obtained:

Table 1. Discharges Obtained with Different Formulas for an Example of a Submerged Orifice

h (m)	0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
Q_s (m^3/s) (Lencastre, with $c_{d1}=c_{d2}=c_{d3}=0.88$)	4.77	4.71	4.56	4.30	3.90	3.38	2.76	1.95	0
Q_s (m^3/s) (according to Eq. (4))	4.77	4.49	4.19	3.86	3.49	3.07	2.56	1.87	0

$$\frac{Q_s}{Q_f} = \sqrt{-5.19\left(\frac{h}{H}\right)^2 + 5.71\frac{h}{H} - 0.57} \tag{7}$$

valid from $h/H = 0.55$ to $h/H = 0.92$. Eq. (7) is similar to the equation of Varshney and Mohanty (1973) but considers new data and is more conservative. Eq. (4) is more conservative yet, as Fig. 1 shows.

It should be noted that Eq. (4) is obtained from prototype data, which means it includes upstream and downstream tridimensional effects. Both equations can be used, depending on safety exigencies. In important works, a physical model should be employed.

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The authors would like to thank the discussers for bringing up some solid arguments and improving the understanding of our manuscript in the process.

We agree with Professors Zhang, Xu, and Wang that y_c cannot be greater than H for free weir flow. We also agree that when one uses 0.91, 0.28, and 0.30 for the c_1, c_2, and c_3 coefficients in Eq. (11), one assumes the flow is free weir, $y_c > H$. The reason for this alleged discrepancy is that in our generalized equation calibration and validation (Fig. 11), from which the coefficients c_1, c_2, and c_3 were derived, free weir flow data were not available because this flow condition did not occur at the selected sites. It is important to remember that the coefficients c_1, c_2, and c_3 depend on the physical characteristics of the structures and the types of flow conditions that the structures experience and can vary from one group of structures to another. With the field flow data we used in our manuscript, we did not have prototype structures that experience all four flow conditions. Therefore, our emphasis on calibrating and validating the generalized equation is on the flow conditions that occurred at the structures, i.e., controlled submerged and uncontrolled submerged conditions in this case. If all four flow conditions are present, Eq. (11) will still be applicable; however, the coefficients c_1, c_2, and c_3 will be different from 0.91, 0.28, and 0.3, respectively. Please also note that the coefficients c_1, c_2, and c_3 were determined from measurements of headwater stage, tailwater stage, gate opening, and discharge at a group of structures with similar physical characteristics. The systematic errors in these measurements were reflected in the coefficients.

Regarding second point made by Professors Zhang, Xu, and Wang, when $h < 0$, it should be set to zero in order to use Eq. (11).

The rest of our closure focuses on the discussion by Drs. Alves and Martin. We agree that the free orifice discharge coefficient of 0.614, derived from our Eq. (11) when one assumes $G_o \ll H$, is reasonable. In the discussers' case of a spillway with sill $L = 1$ m, $G_o = 1$ m, and $H = 2$ m, an unreasonably high discharge coefficient of 0.88 is obtained by using Eq. (11), because the coefficients c_1, c_2, and c_3 determined from the Kissimmee River spillways may not be applicable to the spillway in this example. Regarding the free weir example, the comments made above regarding the calibration and validation of Eq. (11) are applicable.

Discussion of "Energy Dissipation on Block Ramps with Staggered Boulders" by Z. Ahmad, N. M. Petappa, and B. Westrich

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The authors investigated the energy dissipation performance of block ramps for different boulder arrangements, and their experimental results showed that the block ramps could be used as energy dissipators in river engineering applications. The authors selected h_c/H as the main parameter in the study, where h_c = critical depth and H = ramp height. But the discusser argues that h_c/H could not reflect the flow dynamics because when the boulder structure is totally submerged, its influence on flow characteristics will be weakened. The discusser suggests h_c/D_B as the main parameter, where D_B = height of the macroroughness element. Sadeque et al. (2005) found that maximum bed shear stress at a nonsubmerged object was nearly two times higher than at a deeply submerged object. Furthermore, at stepped channels, which could be regarded as series of macroroughness elements, the flow regime is defined based on h_c/D_B, which controls the energy dissipation (Vischer and Hager 1995). Moreover, for baffled aprons, which are closely related to block ramps, Peterka (1983) recommends that the height of macroroughness elements should be 80% of the critical depth of design discharge ($h_c/D_B = 1.25$) to ensure effective energy dissipation.

Furthermore, uniform-flow conditions cannot be achieved in staggered boulder arrangements when the boulders are not submerged in the flow. Boundary-layer separation occurs behind each macroscale element and a logarithmic velocity distribution will not be valid under these conditions (Kucukali and Cokgor 2008). Such a structure cannot be regarded as a block ramp for the protruding flow conditions, and making a distinction between submerged and protruding flow conditions would be meaningful (Kucukali and Cokgor 2007). In addition, the geometric shape of boulders used in the authors' study, such as hemisphere, is questionable because boulders found in nature generally have irregular shapes (Bunte and Abt 2001).

The authors assumed that the effect of Froude and Reynolds numbers could be neglected. They did not present these dimensionless numbers for the flow conditions, yet they investigated the effect of $h_c = \sqrt[3]{q^2/g}$, where q=unit discharge and g=gravitational acceleration. Because $\text{Re}=q/\upsilon$, where Re =Reynolds number and ν=water kinematic viscosity, is it an appropriate approach to neglect the effect of the Reynolds number even though they investigated the effect of h_c, which is a function of q (Kucukali 2007)? Additionally, the presentation of Froude and Reynolds numbers could allow engineers to extrapolate the results to river engineering operations at large Reynolds numbers.

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