DISCUSSIONS AND CLOSURES

Discussion of "ANN and Fuzzy Logic Models for Simulating Event-Based Rainfall-Runoff" by Gokmen Tayfur and Vijay P. Singh

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The authors are commended on developing artificial neural network (ANN), fuzzy logic (FL), and kinematic wave approximation (KWA) models for predicting peak discharge and runoff hydrograph. This discussion focuses on the runoff hydrograph prediction and the goodness-of-fit measurement.

Fig. 8 shows comparisons of simulated hydrographs to observed hydrographs for two testing runs. The goodness of fit between the simulated and the observed hydrographs are assessed using three error measurements: (1) the root-mean-square error (RMSE); (2) the mean absolute error (MAE); and (3) the coefficient of efficiency (E). Their values are presented in Table 2. For the three models and the two runs, the respective values of RMSE, MAE, and E are all close to each other, and because of this, the paper does not indicate which model is better. Since the values of RMSE and MAE are relatively small and the values of E are close to 1.0, the paper concludes that the simulations by all three models are satisfactory.

In addition to the error measurements, another method to assess the goodness of fit is to compare the simulated with the observed hydrographs graphically, as in Fig. 8. This graphical assessment shows that hydrographs simulated by the ANN model follow the general trend of the observed hydrographs. The hydrographs simulated by the FL show that their rate of increase in discharge is sometimes incompatible with that in the observed hydrographs (i.e., the rate of increase may be steep in the FL hydrograph, but it is gentle in the observed hydrograph, and vice versa). It would be interesting to find out why this is so. For the hydrographs simulated by the KWA, the increase in discharge from zero to equilibrium is very steep as compared to that of the observed hydrographs. In fact, for the simulated hydrographs, the rate of increase is three times or more greater than that of the observed hydrographs. It would also be interesting to find out why this is so. With this graphical assessment, it can be concluded that the hydrographs simulated by the ANN model are the best and those by the KWA model are the worst.

In view of the preceding, the discusser asserts that the most important criterion for assessing the goodness of fit should not be the error measurement; instead it should be the comparison of the simulated to the observed hydrographs. Since all the differences between the simulated and the observed hydrographs are summarized in the single error measurement, the intricate differences between the hydrographs are not obvious. *The discusser therefore recommends that any error measurement be used as supplementary information only. The principal criterion for assessing the goodness of fit is the graphical comparison of the simulated with the observed hydrographs.*

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We would like to thank Dr. Wong for his interest in and thoughts on our analysis of runoff hydrograph prediction and the goodness-of-fit measurement. We agree that visual comparison of simulated and measured hydrographs is an important indicator for assessing the performance of models. Visual inspection allows one to see intricate differences between hydrographs. In addition, we think that error measurements are beneficial when quantitatively analyzing the performance of models, as is customary in hydrological and/or hydraulic literature. In our analysis, we in fact employed both measures for comparison. In Fig. 8, we presented simulated and measured hydrographs and commented on the performance of models in simulating the increase and equilibrium of the hydrographs in the subsection "Comparison of Modeling Results." In the same subsection, we commented on the performance of models using computed error measures, i.e., MAE, RMSE, and E. Analysis of the results of comparison, as Dr. Wong points out, indicated that ANN models simulated both the increase and equilibrium of hydrographs quite satisfactorily, with minimum error

Table 1. Rainfall Intensities and Corresponding FL Computed Runoff Rates

Rainfall rate R [mm/h]	Runoff rate Q [L/min]	ΔQ [L/min]
5	13.5	
10	16.4	2.9
15	18.3	1.9
30	21.7	3.4
40	32.8	11.1
45	35.8	3
70	51.5	15.7
74	54.3	2.8

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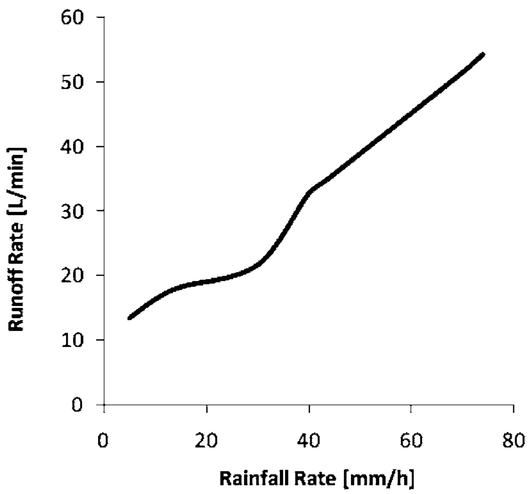

Fig. 1. Runoff rate versus rainfall rate

measurements. However, as Dr. Wong points out, we did not elaborate on the details of the trends of the hydrograph simulations. In particular we did not explain why the KWA model reached equilibrium quickly or why the FL model, at some parts of the hydrographs, had steep increases in the rate of runoff. We comment on these points as follows:

The KWA model reached equilibrium very steeply because the infiltration component of the model employed constant infiltration rate. Tayfur et al. (1993) solved sheet overland flows over infiltrating surfaces and employed the time-dependent Green-Ampt infiltration model, where the model-simulated runoff reached equilibrium gradually. In Fig. 5 of Tayfur et al. (1993), one can see that the KWA model performed as well as the full St. Venant equations in simulating the runoff hydrograph from the S3R2A experimental plot. Also, on the same figure, the runoff hydrograph rose gradually in the case of all models, including KWA. This clearly implies that when the time-dependent infiltration rate model was employed in the KWA model in the current study, the model simulation of the increase could be gradual, rather than steep, and the KWA model could show better performance for simulations in Fig. 8 [see Fig. 8(a) in the current study and Fig. 5 in Tayfur et al. (1993)]. Hence, to avoid any bias in the performance analysis of the ANN, KWA, and FL models in the current study, the reader should be aware that the KWA model employed in the current study used a constant infiltration rate.

As the discusser points out, the simulation hydrographs by the FL model show sudden rises in the runoff rate while reaching equilibrium. This is owing to the nature of the fuzzy logic algorithm that employs fuzzy rules and fuzzy subsets for input-output variables. As seen in Figs. 7 and 9, all variables, aside from rainfall [Fig. 7(b)], which had fuzzy subsets, had subsets with varying base widths. That means that when consecutive input variable values are fed into the system, they may fall into different subsets, depending upon the magnitudes, and trigger different rules from the fuzzy rule base. Consequently they may yield quite different combined fuzzy output sets. This naturally will result in steep rises (or falls) in the trend. We can illustrate this by a simple example: Let us consider two variables of rainfall rate (input variable) and runoff rate (output variable). Let us have the same fuzzy subsets employed in the current study [Fig. 7(b and c)] for the input and output variables, and let us also employ the following five reasonable simple fuzzy rules:

Rule 1: IF rainfall is very low, THEN runoff is very low.
Rule 2: IF rainfall is low, THEN runoff is low.
Rule 3: IF rainfall is medium, THEN runoff is medium.
Rule 4: IF rainfall is high, THEN runoff is high.
Rule 5: IF rainfall is very high, THEN runoff is very high.

Table 1 shows some input rainfall rates (r), FL computed runoff rates (Q), and unit increases in the predicted runoff rates (ΔQ). As seen in Table 1, the increase in the FL computed runoff rate from $r=15$ mm/h to $r=30$ mm/h is slight ($\Delta Q = 3.4$ L/min). However, the increase in the FL computed runoff rate from $r=30$ mm/h to $r=45$ mm/h is significant ($\Delta Q = 14.1$ L/min). This clear trend is also shown in Fig. 1 for better visualization. As seen, there is a slight increase in the steepness when rainfall is increased from $r=15$ mm/h to $r=30$ mm/h. The reason that there is a slight difference in runoff rates for the first case is because $r=15$ mm/h is a member of the low subset with 50% membership, and $r=30$ mm/h is a 100% member of the low subsets. Therefore, both produced values around $Q=20$ L/min corresponding to the low subset of runoff, as a result of the triggered Rule 2. On the other hand, $r=45$ mm/h is almost a 100% member of the medium subset of rainfall, triggering Rule 3 and resulting in a value around $Q=40$ L/min, corresponding to the medium subset of runoff. Thus, as seen in Fig. 1, this results in a significant increase in the steepness of the simulation hydrograph.

Note that in Fig. 8(a and b), the infiltration rate is constant, which is equal to about 19 mm/h, corresponding to the high subset in Fig. 7(a). Similarly, for each experiment, the rainfall intensity is constant, which is about 79 mm/h, corresponding to medium and high subsets in Fig. 7(b). Therefore, in Fig. 8, the change in steepness of the rising hydrographs of runoff rate takes place as a result of the fuzzy subsets of the time variable in Fig. 9. As seen from the figure, the subsets have significantly different base widths, and consequently consecutive entries for the time variable trigger different rules in Table 4 and thus result in milder and steeper trends in the increase of hydrographs.

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Discussion of "Auguste Graeff: Dam Designer and Hydraulic Engineer" by Willi H. Hager and Corrado Gisonni

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The discusser congratulates the authors for their important article. Auguste Graeff is rarely acknowledged for his true contribution to hydraulic engineering and structures. Herein the discusser wishes to add further information on Graeff's coworker Emile Delocre, on the influence of their work in France and overseas, and on several curved gravity dams built around Saint-Etienne based upon the Gouffre d'Enfer Dam design.

Emile Delocre (1828-1908) was a professional French engineer (ingénieur) from the Administration des Ponts et Chaussées (France). He and Auguste Graeff (1812-1884) developed a method to design masonry gravity dams that was first applied to the Gouffre d'Enfer Dam (Delocre 1866). Their work adopted the idea and analysis of the French engineer Augustin de Sazilly (1812-1852). The design method is considered the basic method of analyzing the stability of gravity dams. The Gouffre d'Enfer Dam (1866) is a curved gravity dam located on the Furan River upstream of the city of Saint-Etienne, France [Fig. 1(a)]. It is sometimes called the Furan Dam or Furens Dam (Humber 1876; Smith 1971), but locally it is known as the barrage du Gouffre d'Enfer.

The work of Graeff and Delocre had an immediate influence on dam and spillway designs in France and overseas. In his 1876 book, the English engineer Humber was appreciative of the French savoir-faire: "The theory of masonry dams forms the subject of a very interesting and rather elaborate memoir by Delocre of the Administration des Ponts et Chaussées" (Humber 1876, p. 123). The reputation of the Gouffre d'Enfer Dam was known to him: "The dam of the Furens reservoir, as actually constructed, is shown on Fig. 86" (p. 125). The writings of Humber suggest that the works of de Sazilly, Graeff, and Delocre were well known in the United Kingdom, where it was indeed extended by W. J. M. Rankine (1820-1872) (Rankine 1872). The expertise of Graeff and Delocre also influenced Australian dam designers (Chanson and Whitmore 1998; Chanson and James 2002). For example, the curved gravity dam of Lower Stony Creek (1873) was Australia's first curved gravity dam, and its design was based upon the works of Graeff, Delocre, and Rankine (Sankey 1871; Dobson 1866;

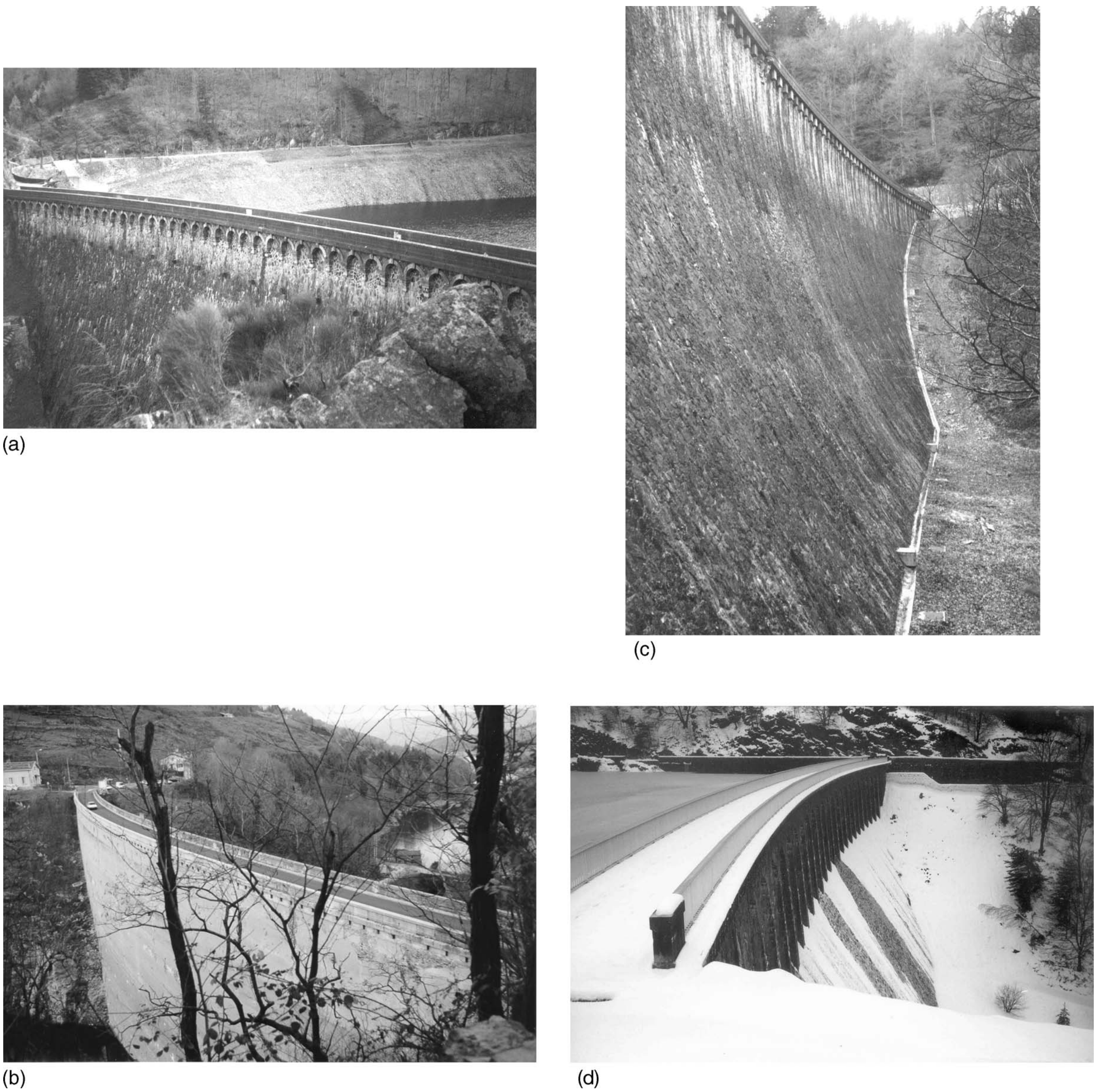

Fig. 1. French curved gravity dams in the region of Saint-Etienne (a) December 3, 1994, photograph of the Gouffre d'Enfer Dam (masonry dam completed in 1866): H=60 m, L=102 m. View of the downstream face from the left bank. (b) December 3, 1994, photograph of the La Rive Dam (masonry dam completed in 1870): H=46.3 m, L=165 m. View of the upstream face from the right bank: the reservoir was empty at the time because of structural concerns. (c) December 3, 1994, photograph of the Pas-du-Riot Dam (masonry dam completed in 1873): H=36 m, L=154 m. View of the downstream face from the right bank. (d) January 1997 photograph of Le Cotatay Dam (masonry dam completed in 1901): H=43 m. View from the right bank.

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Fig. 2. January 30, 2000, photograph of the Lower Stony Creek Dam in Geelong, Australia (masonry dam completed in 1873): H =18.3 m, L=68 m. The dam was the first curved gravity dam in Australia. Note the low-flow spillway section in the middle of the dam crest.

Harper 1998). Fig. 2 shows a recent photograph of the Lower Stony Creek Dam.

A number of masonry dams were subsequently built in the region of Saint-Etienne as replicas of the Gouffre d'Enfer Dam (Table 1, Fig. 1, and Fig. 3). The Ternay Dam was completed in 1867 to supply water to the city of Annonay and its factories. (Annonay is about 27 km southeast of Saint-Etienne.) The La Rive Dam, also called Le Ban Dam and completed in 1870, was designed to supply water to the city of Saint-Chamond, 5 km northeast of Saint-Etienne. The Pas-du-Riot Dam (1873) is located immediately upstream of the Gouffre d'Enfer Dam to supply water to the city of Saint-Etienne. Other curved gravity dams include L'Echapre, L'Ondenon, Le Cotatay (all near Saint-Etienne), La Tâche near Roanne, and Le Joux near Tarare (Table 1). Some details on these dams are listed in Table 1 including the dam wall thickness at the base and at crest. Interestingly, all these curved gravity dams were equipped with stepped cascade spillways on the left or right hillsides. Most have some unlined rock cascades, but the Pas-du-Riot Dam was equipped with a masonry-

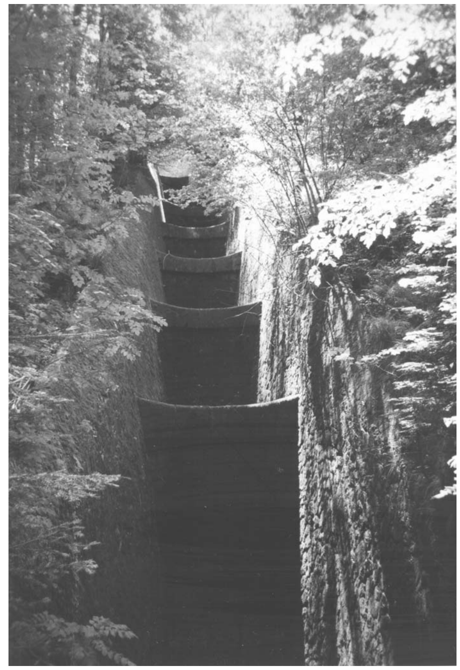

Fig. 3. June 1998 photograph of masonry-stepped spillway of the Pas-du-Riot Dam

stepped spillway (Fig. 3). This spillway is interesting by modern standards, and the discusser noted that large timber, debris, and logs passed successfully over it. The spillway capacity was recently enlarged, but the steep, stepped spillway remains untouched.

The discusser inspected the dams listed in Table 1 between 1994 and 1998. Figs. 1 and 2 present some photographs of the structures. At that time, the La Rive Dam still stood but was no longer in use (Fig. 1(b)). The structure had been deemed unsafe, and a large tunnel outlet had been drilled through the dam in

Table 1. Characteristics of French Curved Gravity Dams Built around 1860–1870

Name	Construction period	Dam height (+) (m)	Crest length (m)	Crest thickness (m)	Base thickness (m)	Comments
Le Gouffre d'Enfer	1862–1866	60	102	3.15	49	Still in use. Spillway refurbished in the 1990s.
Ternay	1867	41	—	—	—	Near Annonay. Still in use. Dam wall and spillway refurbished in the 1990s.
La Rive	1866–1870	48.3	165	5	35	
Pas-du-Riot	1873	36	154	—	18.6	Still in use.
La Tâche	1891	51	221	4	47.5	Near Roanne. Still in use. Refurbished in the 1990s.
L'Echapre	1894–1898	36.5	165	3.9	24.54	Near Firminy. Still in use.
Le Cotatay	1889–1901	43	118	4.6	33.35	Near Chambon-Feugerolles. Still in use.
L'Ondenon	1901–1904	32.6	15	4.3	29.6	Near La Ricamarie. Still in use.
Le Joux	1901–1904	25	126	4	20	Near Tarare. Dam wall refurbished and raised in 1951. Still in use.

Notes: (+) measured above foundation; (−) not available.

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1994-1995 to prevent the water level from rising upstream of the masonry structure. Since that time, the dam wall has been reinforced, and the reservoir is in use again.

Altogether, Graeff and Delocre had a significant influence on gravity dam design in France and overseas.

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The writers appreciate the interest of Professor Chanson in our historical work. The biography of Emile Delocre (1828-1908) is presented in Hager (2008), and that of Graeff is available in Hager (2003). Their lives and their impact in dam design are therefore well described. Chanson's photos add to our original work.

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