of a number of these devices in specific nodes of the network and rating them to the appropriate pressures, with the objective of reducing the class of the downstream pipes.

The first step in the proposed method is to determine the size (and consequently the cost) of each possible PRV that can be located in the network nodes; therefore, the sizes of the PRVs that appear in the final solution are previously specified and known prior to the selection process. So, the basic data for the sizing of a given PRV (maximum flow and maximum head loss) are known when the network design problem is established. The optimization procedure does not directly size PRVs, but consists in the selection of the network nodes where a PRV will be installed. For given requirements of maximum flow and maximum pressure drop, it will be always possible to find a PRV model or some arrangement to accomplish these requirements.

As far as the possible operational problems caused by intermittent flows are concerned, they are solved in practice by using different closure devices (as the V-port); with a great difference between maximum and minimum flow, it is possible to use a parallel arrangement, with two or several PRVs of different sizes, that allow a right operation in a wide range of flows (Mateos 1990).

A PRV must not impede the filling of the pipes; there are different models of pilot control PRV that allow the rapid filling of the pipes by acting only when the pipe is full.

Regarding the transients caused by a PRV, they can be reduced by varying the maneuvering speed of the valve; in the case of grouped PRVs, one can set a different pressure rate for each PRV to avoid its simultaneous operation.

The proposed optimization method uses PRVs as an auxiliary element in the economic design of networks. It is possible to use other additional devices, such as air-release valves for helping in the filling of the pipes or pressure-sustaining valves to avoid a rapid emptying of the pipes; in any case, this type of valve is not considered part of the optimization process.

The discussers also proposed the application of the method for determining the most efficient set of PRVs in a main of prestressed concrete, 40 km long. In that system, the proposed method can be applied first to analyze possible PRVs in intermediate nodes of the main, then comparing their cost with the reduction in cost of the downstream main due to the effect of the PRVs. With this information, the method allows to obtain the set of PRVs that gives the maximum economic efficiency. Once the system and the set of PRVs are sized, it will be possible to use additional devices to provide safety to the system.

Finally, the writers wish to thank to Juan Angel Serrano for his useful comments about the operational aspects of PRVs.

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ENERGY DISSIPATION ON STEPPED SPILLWAYS[a]

Discussion by Hubert Chanson[2]

The author provided interesting data on stepped spillway flows. The writer would like to add some information on flow resistance of skimming flows and discuss the energy dissipation on stepped chutes. It will be shown that the author's results are not dissimilar to results previously obtained by other researchers (Table 2).

FLOW RESISTANCE OF SKIMMING FLOWS

For skimming flows, horizontal vortices develop beneath the pseudo-bottom formed by the external edges of the steps. The vortices are maintained through the transmission of turbulent shear stress between the skimming stream and the recirculating fluid underneath.

If uniform flow conditions are reached along a constant slope channel, the friction factor can be deduced from the momentum equation as (Chanson 1993):

$$f = \frac{8 * g * \sin\theta * y_0^3}{q_w^2} * \frac{D_H}{4} \tag{4}$$

[a]May, 1993, Vol. 119, No. 5, by George C. Christodoulou, (Paper 4137).

[2]Lect., Dept. of Civ. Engrg., Univ. of Queensland, Brisbane QLD 4072, Australia.

TABLE 2. Characteristics of Model Studies

Reference (1)	Slope (degrees) (2)	Scale (3)	Step height h (m) (4)	Number of steps (5)	Discharge q_w (m²/s) (6)	Remarks (7)
Sorensen (1985)	52.05	1/10	0.061	11	0.006–0.28	Monksville dam spillway
Sorensen (1985)	52.05	1/25	0.024	59	0.006–0.28	model (United States) W = 0.305 m
Bayat (1991)	51.3	1/25	0.024, 0.03, 0.02	—	0.006–0.07	Godar-e-landar spillway model (Iran), W = 0.3 m
Diez-Cascon et al. (1991)	53.1	1/10	0.03–0.06	50–100	0.022–0.28	Spain, W = 0.8 m
Stephenson (1991)	54.5	—	—	—	—	Kennedy's value model (South Africa)
Peyras et al. (1992)	18.4, 26.6, 45	1/5	0.20	3, 4, 5	0.04–0.27	Gabion stepped chute (France), W = 0.8 m
Bindo et al. (1993)	51.34	1/21.3	0.038	31–43	0.01–0.142	M'Bali spillway model
Bindo et al. (1993)	51.34	1/42.7	0.019	43	0.007–0.04	(France), W = 0.9 m

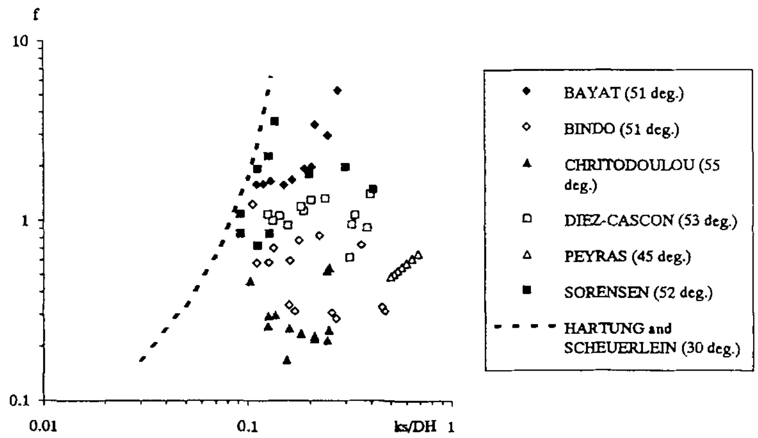

FIG. 5. Friction Factor of Skimming Flow

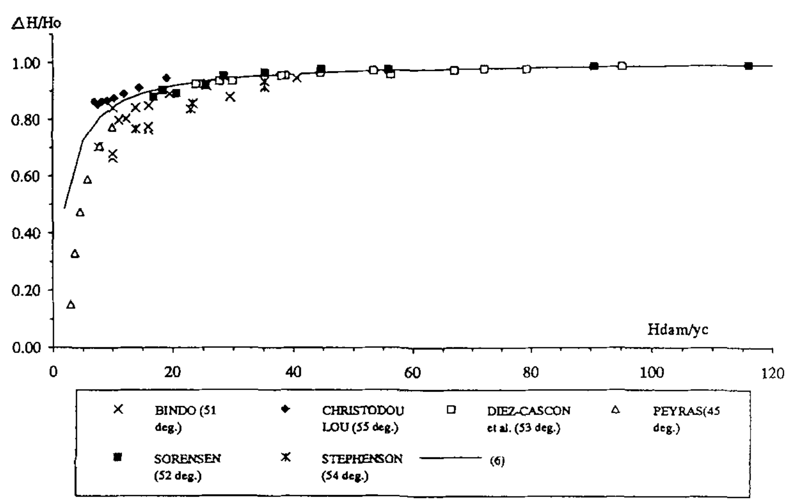

FIG. 6. Energy Dissipation in Skimming-Flow Regime

where f = the friction factor ($f = 4^*C_f$); q_w = the discharge per unit width; and D_H = the hydraulic diameter. For nonuniform gradually varied flows, the friction factor can be deduced from the energy equation:

$$f = \frac{8^*g^*y^2}{q_w^2} * \frac{D_H}{4} * \frac{\Delta H}{\Delta s} \quad (5)$$

where ΔH = the total head loss over a distance Δs; and $\Delta H/\Delta s$ = the friction slope.

The author reanalyzed model data using (4) and (5). Details of the flow conditions are reported in Table 2. The results are presented in Fig. 5, where the friction factor f is plotted as a function of the relative roughness k_s/D_H [roughness k_s is defined in Chanson (1993) (i.e., $k_s = h^* \cos\theta$)].

For channel slopes ranging from 50° to 55°, Fig. 5 shows a large scatter of friction-factor values observed on various models. An analysis of all the data indicates no correlation between the friction factor, the Reynolds number, and the relative roughness. The author's data are within the scatter of other data.

Fig. 5 also presents results obtained for flows over rockfilled channels for a 30° slope (Hartung and Scheuerlein 1970). The results indicate friction factors of similar order of magnitude as the results obtained on stepped spillways.

It must be emphasized that the data were analyzed neglecting the effects of air entrainment. No information is available on the amount of air entrained during these experiments.

ENERGY DISSIPATION

In skimming flow, most of the energy is dissipated in the maintenance of stable depression vortices (Rajaratnam 1990). If uniform flow conditions are reached at the downstream end of the spillway, Chanson (1993) showed that the total head loss can be rewritten in terms of the friction factor, the spillway slope, the critical depth, and the dam height:

$$\frac{\Delta H}{H_0} = 1 - \frac{\left(\frac{f}{8^* \sin\theta}\right)^{1/3} * \cos\theta + \frac{1}{2} * \left(\frac{f}{8^* \sin\theta}\right)^{-2/3}}{\frac{H_{dam}}{y_c} + \frac{3}{2}} \tag{6}$$

where H_{dam} = the dam height ($H_{dam} = N^*h$). Fig. 6 compares (6) with model data. Eq. (6) was computed for $\theta = 55°$ and $f = 1.30$ as used by Chanson (1993). Eq. (6) indicates that the energy-loss ratio increases with the height of the dam. That trend, also observed on Fig. 4, is not unexpected but was demonstrated previously by Stephenson (1991) and Chanson (1993). Fig. 6 shows a good agreement between the experimental data and (6). Again the author's data are within the scatter of the other model data.

It must be emphasized that (6) critically depends on the estimation of the friction factor. Fig. 5 shows a large scatter of friction-factor values observed on various models. Chanson (1993) showed that the friction factor and the rate of energy dissipation are affected significantly by the rate of aeration. Therefore (6) must be used with caution.

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APPENDIX II. NOTATION

The following symbols are used in this discussion:

D_H = hydraulic diameter (m);
f = Darcy friction factor;
H_{dam} = dam height (m);
k_s = roughness height (m) or step dimension normal to the flow; $k_s = h^* \cos\alpha$;
q_w = water discharge per unit width (m^2/s); and
s = distance (m) along the channel from crest.

Discussion by Sandip P. Tatewar[3] and Ramesh N. Ingle[4]

The author has described interesting observations of energy loss on stepped spillways based on laboratory experiments and attempted to establish the relation for energy loss in terms of

[3]Lect., College of Engrg., Amravati, India.
[4]Prof. of Civ. Engrg., Visvesvaraya Regional College of Engrg., Nagpur—440 011, India.

number of steps N; the geometry of steps h and l; and the discharge, which is expressed in terms of critical depth y_c. Based on experimental results, the author obtained a relationship between $\Delta H/H_0$ and y_c/Nh and presented it in Fig. 4. He has assumed the variable l/h as constant at 0.7, which is equal to the slope of the downstream spillway face in his experiments. The observations reported are for $N = 10$ and 13, and out of these, the first seven steps were on the curved portion of the spillway, for which l/h is not constant. This means l/h varied over the majority of the spillway face, and its effect on energy loss, which can be considerable, has been overlooked. Furthermore, the variable Nh is equal to the vertical distance between the crest of spillway and tail-water level, which is nearly equal to the height of the dam, Z. The two sets of experimental observations are for $N = 10$ and 13. The height of steps h is kept the same for both sets of observations. Therefore, a change in Nh corresponding to $N = 10$ and $N = 13$ is equivalent to change in dam height and would indicate the effect of y_c/Z on energy loss. Furthermore, the ratio of the number of steps on the curved portion to the number of steps on the constant-slope portion in the two sets of observations is 7/3 and 7/6, respectively. This means the contribution to energy loss by steps on the curved portion is more than that by steps on the constant-slope portion, a situation unlikely to occur in a prototype spillway. Therefore, the writers are of the opinion that the relationship given by the author in Fig. 4 may not be directly applicable to prototype spillways.

EXPRESSION FOR ENERGY LOSS

Assuming the flow over constant slope portion of spillway face as uniform flow in a wide rectangular channel, the energy loss can be expressed as

$$\Delta H = \frac{C_f l v^2}{2 g y_0} \tag{7}$$

Substituting the value of v in terms of g, y, and y_c, and H_0 in terms of ΔZ, Y, and $v_0^2/2g$, the energy-loss ratio can be expressed as

$$\frac{\Delta H}{H_0} = \frac{\frac{1}{2} C_f l \left(\frac{y_c}{y_0}\right)^3}{\Delta Z + Y + \frac{v_0^2}{2g}} \tag{8}$$

Furthermore, using the Ogee weir formula to express discharge over the spillway in terms of head over the spillway including velocity of approach head, (8) with simple algebraic manipulations can be written as

$$\frac{\Delta H}{H_0} = \frac{\frac{1}{2} C_f l \left(\frac{y_c}{y_0}\right)^3}{\sin\theta \left(1 + C\frac{y_c}{Z}\right)} \tag{9}$$

where $C = (9/8\ C_d^2)^{1/3}$. The value of coefficient of discharge C_d for an open spillway may vary from 0.738 to 0.64, and the corresponding value of C would vary from 1.273 to 1.400. The expression for the ratio of energy loss given in (9) can be further simplified by using (3) as

$$\frac{\Delta H}{H_0} = \frac{1}{1 + C\left(\frac{y_c}{Z}\right)} \tag{10}$$

COMPARISON WITH EXPERIMENTAL RESULTS

Eq. (10) can be used to estimate energy loss on stepped spillways for uniform flow on the constant slope of the downstream face. For the purpose of comparison, (10) is plotted along with the experimental results of the author and of Sorensen (1985) in Fig. 7. It shows good agreement with Sorensen's results for values of y_c/Z up to 0.1 but predicts higher energy loss for the author's results. This may be due to the variation in the value of l/h over a larger portion of the spillway surface in author's experiments. It is well known that the slope of spillway surface has considerable effect on energy loss and its variation has not been considered by the author. Furthermore, experimental results of Bayat (1991) compared to (10) are shown in Fig. 8. Bayat's experimental results are for the same value of dam height and for different step sizes. The effect of step size or number of steps on energy loss is comparatively less, though more steps indicate more energy loss as expected. Eq. (10) compares well with Bayat's results, and indicates the upper limit of energy loss.

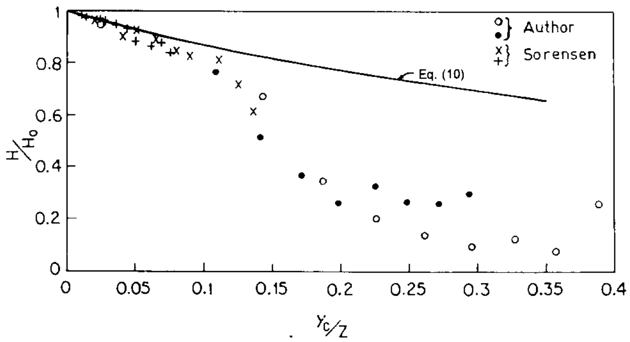

FIG. 7. Variation of Relative Head Loss $\Delta H/H_0$ with y_c/Z

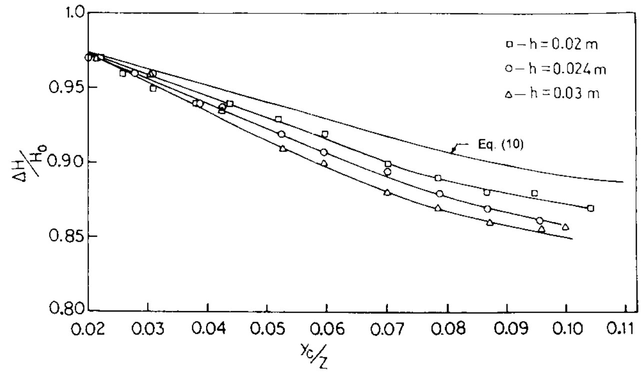

FIG. 8. Comparison of (10) with Bayat's (1991) Experimental Results for Different Step Heights

APPENDIX. REFERENCE

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Discussion by P. Veerabhadra Rao,[5] Member, ASCE

The concept of effectively dissipating kinetic energy by using stepped spillways was proved in field conditions for different dams in the United States, Australia, and South Africa. The design philosophy was also used for other structures [e.g., the Flight Sewer in Baltimore (Babbit 1947)] adopting granite steps. The objective of this discussion is to highlight the flow phenomenon on steps and the results of other investigators.

Fig. 9 shows three-dimensional flow over the Chew Valley spillway in England (*Hydraulic Research Station* 1978), and clearly depicts the physics of flow phenomena over the steps under field conditions. The flows over stepped spillways have been categorized into (1) Nappe and skimming flows (Rajaratnam 1990); and (2) isolated nappe, partial nappe, and skimming flows (Peyras et al. 1992). The flow in Fig. 9 appears to be a transition between partial nappe and sheet flow with aeration. It is also clear from Fig. 9 that the aeration takes place on the first step itself despite the depth of flow $d << h$, and the dissipation of energy appears most effective. The author and Peyras et al. (1992), have also reported high energy dissipation at low discharges from their experimental results.

The effective roughness coefficient, C_f ($= f/4$) reported by Rajaratnam (1990) for stepped structure ranges from 0.05 to 0.18 for his own experimental studies, from 0.11 to 0.20 with an average of 0.18 for moderate flows, and from 0.25 to 0.28 for very small flow rates using the experimental results of Sorensen (1985). The third set of values clearly demonstrates why the energy dissipation appears maximum in Fig. 9 when the flow rates are small. In fact, it was clearly stated as early as 1970 that the advantage of steps is that the energy is dissipated a little at a time but this is true only at low flow rates (White 1970). Further, Noori (1984) studied in detail stepped steep open channel flows and reported a drag coefficient of 0.19 for $h/l = 0.2$ and $N = 62$ [$(d + h/2)/h > 6$] and of 0.17 for $h/l = 0.1$ and $N = 100$ [$(d + h/2)/h > 10$]. In these studies, maximum number of steps is used though the slopes are gradual compared to the actual Waterways Experimental Station spillway profiles. It appears that effective roughness coefficient and drag coefficient are the same in this type of experimental study. Noori (1984) reported that drag coefficient $C_D = f(d/h$, Froude number, Reynolds number, and $h/l)$, which appears correct from the data of Rajaratnam (1990) and Sorensen (1985). Noori's (1984) experiments further depict that Froude number is a dominant factor in determining drag coefficient, which increases with channel slope. It is also observed that the drag coefficient varies with $(d + h/2)/h$ up to a certain value, after which it becomes constant. The author's comments are requested on these specific observations.

[5]Deputy Chf. Engr. (R&D), Res. and Devel. Group, Tata Consulting Engrs., 73/1 St. Mark's Road, Bangalore 560 001, India.

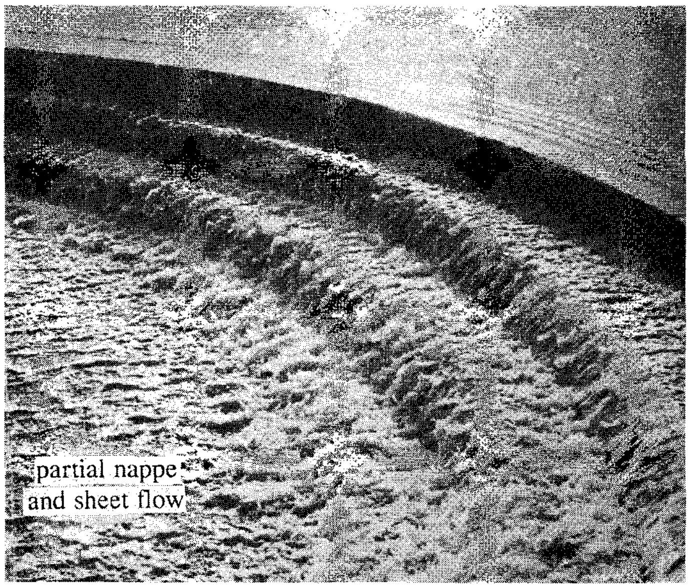

FIG. 9. Flow over Steps for Chew Valley Spillway (Reproduced with Permission of Controller of Her Majesty's Stationery Office, London)

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Closure by George C. Christodoulou,[6] Member, ASCE

The discussions by Chanson, Veerabhadra Rao, Tatewar, and Ingle raise several interesting points and demonstrate the importance of research on stepped spillways. The complexity of the flow phenomena occurring on such structures presents a challenge for further studies. Naturally, most of the discussers' comments focus on the effective resistance of the stepped surface and the consequent estimates of energy loss.

Chanson presents an expression for the friction coefficient [(4)] that essentially coincides with (3), taking into account the fact that $f = 4c_f$ and that, for $B >> y$, $D_H = 4y$. The writer has already suggested that the steps act as a macroroughness, and Chanson's definition of $k_s = h \cos \theta$ seems a pertinent measure of such roughness. As Veerabhara Rao points out, based on Noori's (1984) analysis, the effective friction of the stepped surface varies with the parameter $(d + h/2)/h$ up to a certain value, then becomes constant. Indeed, that parameter may be rewritten as:

$$\frac{D}{h} = \frac{\left(d + \frac{h}{2}\right)}{h} = \frac{d}{h} + \frac{1}{2} = \frac{d \cos \theta}{h \cos \theta} + c = \frac{y}{k_s} + c \tag{11}$$

where c = a constant. Therefore, Noori's (1984) observation reflects the fact that the effect of the steps depends on the relative macroroughness k_s/y (or k_s/D_H); this effect is expected to diminish as y/k_s becomes large, i.e., k_s is small compared to the depth of flow.

The comparison of friction factors derived from various studies and presented by Chanson (Fig. 5) shows considerable internal consistency within each study but significant deviations between studies. The large overall scatter seen in the plot of friction factor versus relative roughness may be due, in the writer's opinion, to the following reasons:

1. The friction factor depends not only on the relative roughness but on other parameters as well, notably the Reynolds number (as is the case for flow in pipes, boundary

[6] Assoc. Prof., Dept. Civil Engrg., Natl. Tech. Univ. of Athens, Athens 15773, Greece.

layers, and so on). Since all data used originate from relatively small-scale laboratory models, the effect of the Reynolds number could be significant.

2. The flow is not uniform in several of the data, especially those of the low spillways. Eqs. (3) and (4) are based on uniform flow considerations, and the use of the local depth y of an accelerating flow instead of y_0 leads to underestimation of friction factors compared to those corresponding to uniform conditions.
3. The effect of aeration is not taken into account in most studies but it may be important, as Chanson noted.

It is because of the aforementioned inherent difficulties in the interpretation of experimental data that the friction coefficients computed in the paper were considered to be merely order-of-magnitude approximations and were not subsequently used for a "theoretical" estimate of head loss. The analytical expression proposed by Chanson for the energy loss [(4)], as well as previous equations (e.g., Rajaratnam 1990; Stephenson (1991) are subject to the stated limitations, notably the uniformity of flow. Indeed, as seen in Fig. 6, Chanson's formula fits the experimental data better for large dam heights, where presumably uniform flow was attained.

The writer agrees with both Chanson and Veerabhadra Rao that the dependence of head loss on the total height of the dam is to be expected, in the sense that the loss on a given stepped surface increases with the length of the surface. It turns out that the total height is essentially the most important parameter, but only as long as the step characteristics are roughly the same. That is, no comparison should be made on the basis of total height between structures with widely different steps (e.g., in the limit $h/l \rightarrow 0$, smooth spillway). Besides, as seen in Fig. 3, the importance of the number of steps and therefore the total height becomes appreciable with increasing discharge but is minor at low discharges, where very high dissipation ratios may occur over even a few steps; it is precisely in the range of low discharges (small y_c/h) that the stepped spillway proves most effective.

Concerning Veerabhadra Rao's remarks about the dependence of the drag coefficient C_D on the Froude number based on Noori's (1984) results, the writer's opinion is that the reported dependence is due to an erroneous interpretation of the experimental results. In fact, the apparent dependence of C_D on the Froude number is a consequence of having introduced a priori in the dimensional analysis the variable g, which is actually not independent of other variables used under the assumed uniform flow conditions—namely the flow depth expressed by the variable $D = d + h/2$, the slope S, and the velocity V. The unique dependence of C_D on F derived by Noori is very nearly an equivalent expression of uniform flow. This can be seen considering the suggested empirical best-fit expression

$$C_D = \frac{2.24}{F^{2.13}} \tag{12}$$

and observing that his data may also be well described by the equation

$$C_D = \frac{a}{F^2} = \frac{agD}{V^2} \tag{13}$$

where $a \approx 2$. Eq. (13) is essentially a uniform flow expression similar to the Chezy formula, written as

$$C_f = 2g \frac{dS}{V^2} \tag{14}$$

taking into account that $D \approx d$ and that the drag coefficient C_D introduced by Noori is directly related to the friction coefficient C_f with a proportionally constant depending on the slope θ. It is worth mentioning that Noori himself had stated in his paper that in the literature, most other investigators simply dropped the effect of Froude number, reasoning that it has no influence on the resistance.

The comments by Tatewater and Ingle regarding the contribution of the curved part of the crest to the energy loss are correct, strictly speaking. However, the existence of the curved portion with variable l/h ratio is usual in practice; simply its influence on the overall head loss decreases as the height of spillway increases. On the contrary, it may become significant in low to medium-sized spillways, such as the one examined in the paper. Over the curved portion, the flow conditions (accelerating flow with gradual boundary layer development) are significantly different than those on a long uniform stepped surface, therefore neglecting that region would lead to unrealistic results for small and moderate structures.

The proposed (10) seems oversimplified compared to other previously derived expressions for the head loss under the same assumption of uniform flow [Rajaratnam (1990), Stephenson (1991), and Chanson's discussion]. This oversimplification may partly explain the deviation of predicted loss compared to the author's data, since Fig. 6 shows that his formula agrees fairly well with both the author's and Sorensen's data. Even so, Figs. 7 and 8 by Tatewater and Ingle serve to illustrate qualitatively that theoretical estimates based on the uniform flow assumption correspond to an upper limit for energy dissipation attainable for a large number of steps (high spillways), whereas smaller loss ratios should be expected in lower structures where uniform conditions are unlikely to occur.

THREE-DIMENSIONAL FLOW STRUCTURE AT OPEN-CHANNEL DIVERSIONS[a]

Discussion by Stuart J. McLelland,[3] Philip J. Ashworth,[4] and James L. Best[5]

The authors correctly highlight the three-dimensional nature of flow in open channels and its important role in channel diversion dynamics. This discussion questions some of the ideas presented by the authors and suggests additional contributory sources for the development of secondary flow at channel divergences.

DEPTH CHANGES AT OPEN-CHANNEL DIVERSIONS

The authors indicate that in all tests the flow depth was constant at 18.6 cm (page 1224), which is inconsistent with the local changes in pressure and water depth that must occur in zones of flow separation and stagnation. Law and Reynolds (1966) and Lakshmana et al. (1968) demonstrate that the water-surface elevation is lower at the upstream junction corner as a result of the separation zone, and raised at the downstream junction corner in the zone of flow stagnation. The width and length of these zones will determine the distance taken for the flow to return to the depth characteristic of the upstream channel.

The authors also state that the Froude number is greater in the diversion channel than in the main channel (page 1225), which, if measured where the depth is 18.6 cm, implies flow acceleration in the diversion. This is contrary to the U_2/U_1 ratio given (page 1224), which suggests a flow deceleration and highlights the significant changes in depth that must occur in the diversion channel.

The corollary of these downstream changes in flow velocity and depth is that they will alter the magnitude and three-dimensional distribution of the streamwise vorticity (e.g., Bradshaw 1987). Therefore, the downstream variation of flow velocity at channel diversions may be significantly different from that at natural river bends. Flow acceleration will be greatest at the diversion entrance, where the free-stream width is restricted by the separation zone. This will stretch and amplify the streamwise vorticity. The simultaneous reduction in flow depth will increase the velocity gradient which and therefore strengthen the skew-induced vorticity. Subsequent flow deceleration as the separation zone narrows will lead to downstream dissipation of vorticity in the diversion channel.

SEPARATION ZONE

The role of the separation zone in influencing the three-dimensional flow structure is not emphasized by the authors even though the separation zone width and the three-dimensional shape of this region will alter the streamline curvature. The rotation of the streamlines toward the diversion channel will generate skew-induced vorticity. Two elements of the streamline pattern shown in Fig. 3 will enhance the production of skew-induced vorticity: (1) The differences in streamline curvature between the bed and near-surface; and (2) the greater rotation on the outer bank of the diversion. These factors will increase rotation around the vertical axis, which will be further amplified by the vertical velocity gradient (see the preceding). As the separation zone decreases in width downstream, rotation of the flow toward the inner bank will continue to produce skew-induced vorticity. Additionally, entrainment of fluid into the separation zone will promote lateral transfer of flow from the outer to inner banks.

STRENGTH OF VORTICITY

The authors express the strength of secondary circulation as the difference between the transverse velocity near the surface and that near the bed (page 1227). Calculations are presented for a point A (Fig. 1), which is located at the edge of the secondary cell and in close proximity to the stagnation zone. At this position, the vertical velocity component V may be important,

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[a]November, 1993, Vol. 119, No. 11, by Vincent S. Neary and A. Jacob Odgaard (Paper 4759).

[3]Grad. Student, School of Geography and Dept. of Earth Sci., Univ. of Leeds, Leeds, West Yorkshire, LS2 9JT, U.K.

[4]Lect., School of Geography, Univ. of Leeds, Leeds, West Yorkshire, U.K.

[5]Lect., Dept. of Earth Sci., Univ. of Leeds, West Yorkshire, U.K.