Quasi-Two-Dimensional Simulation of Scour and Deposition in Alluvial Channels[a]

Discussion by James P. Bennett,[5] Member, ASCE

In their separate treatment method version of USTARS, the authors have presented a significant conceptual improvement over the technology used in GSTARS. As the authors point out, separate treatment of suspended and bed load allows use of the model in disequilibrium situations where deposition predominates, as in their reservoir and estuarine examples. In addition, the formulation is general and thus applicable both to equilibrium situations, where separate estimates of suspended and bedload are needed, as well as to disequilibrium situations, where erosion predominates. Furthermore, because it incorporates lateral dispersion and thus is capable of transferring suspended sediment from one stream tube to another, the separate treatment method version of USTARS is superior to GSTARS, which does not provide any such capability. However, perhaps because their examples primarily concern disequilibrium sedimentation, the authors have chosen an inappropriate version, (9), of the van Rijn (1984) expression to represent the reference level concentration C_{ek} for the erosion of individual k size fractions of the bed sediment; this is the topic of the present discussion.

The van Rijn (1984) expression for determining the reference concentration C_e at the base of the suspended load layer, at elevation a above the average streambed level, is

$$C_e = 0.015 \frac{D_{50}}{a} \frac{T^{1.5}}{D_*^{0.3}} \tag{33}$$

where $a = \Delta/2$, Δ = amplitude of the dune bed, if this amplitude is known, or $a = k_s$, the equivalent Nikuradse roughness height, if dune amplitude is not available, and a is always greater than 0.01h. Garcia and Parker (1991) found (33) to be one of two of the best of seven such relations in determining the reference concentration for uniform bed sediment; they use a = 0.05h. The authors fail to define the reference height and need to do so in the interest of consistent application of their protocol. In their adaptation, (9), of (33) for computing reference concentrations for individual bed size fractions, C_{ek}, the authors replace D_{50} with D_k and T with T_k. Eq. (9) computes the reference concentration as though all of the bed consists of size fraction k and hence must be multiplied by the bed fraction, β_k, to give the appropriate concentration of size fraction k in the suspended mixture; this is done in the source term, (8). In the case of the substitution of T_k for T, it is unclear from the discussion accompanying (9) whether the term u_{*cr} used in computing T_k is actually for each individual size fraction, k, or for the mixture. The consequences of this uncertainty are relatively minor because, for the sand sizes, u_{*cr} varies relatively little. However, the consequences of the substitution of D_k for D_{50} are more severe, because, as demonstrated below, the reference concentration, C_{ek}, varies more nearly with $1/D_k$ than with D_k. A portion of the data used by van Rijn (1984) in determining the constant of proportionality (0.015) in (33) was presented by Scott and Stephens (1966) and consists of cross-sectional average hydraulic parameters, vertical velocity profiles, and sieve analysis of both bed material and of a series of suspended point sample concentrations collected from a number of verticals over a range of discharges from 1961 to 1963 at a bridge section on the Mississippi River, at St. Louis. Using these data, it is possible to evaluate the equivalent of this proportionality constant, A, in (9) from

$$A = \frac{\chi_k C_e}{\beta_k \dfrac{D_k}{a} \dfrac{T_k^{1.5}}{D_*^{0.3}}} \tag{34}$$

where C_e is the concentration measured at the reference elevation; and χ_k is the measured fraction of the sample in size-range k. If (9) is an appropriate representation of the reference concentration, A should be independent of D. Fig. 18(a) shows the individual values of A, where D_k are taken as the geometric mean diameters for each of the four ϕ ranges in the sieve data. In this plot are included computations only for C_e values from point samples, according to van Rijn's (1984) original definition, as elevations $a \leq \Delta/2$, where the dune amplitude Δ was calculated separately for each sampling vertical using methods slightly modified from Bennett (1995). The transport stages T_k

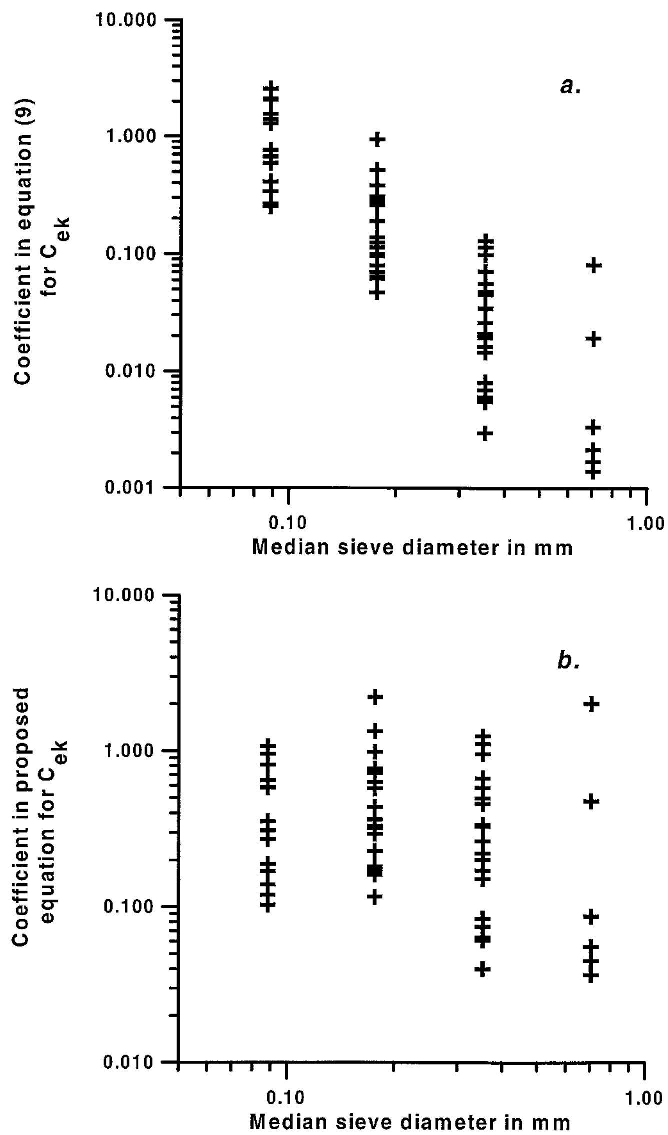

FIG. 18. Variation of: (a) Coefficient *A* of (9) and (34); (b) Coefficient *B* of (35) with Geometric Average Particle Diameter of Sieve Sizes for the Mississippi River Data of Scott and Stephens (1966)

[a]July 1997, Vol. 123, No. 7, by Hong-Yuan Lee, Hui-Ming Hsieh, Jinn-Chuang Yang, and Chih Ted Yang (Paper 1876).

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J. Hydraul. Eng. 1999.125:207-207.

were also computed for the hydraulic and bedform characteristics at each vertical using the methods of Bennett (1995) and u_{*cr} values for the appropriate k'th size fraction. To ensure exclusion of the washload only fractions more coarse than d_3, the 3% finer diameter, of the bed material are included in the plot. Again, the individual values of A should be independent of D, whereas Fig. 18(b) shows a clear trend. Thus the authors' formulation of (9) is inappropriate to its intended purpose.

Trial and error evaluation of the Scott and Stephens (1966) data indicates that a formulation based on (33), and equivalent to (9), which behaves consistently across the sand-size range, is

$$C_{ek} = B \frac{D_{50}}{a} \left[\frac{T}{D_{*k}} \right]^{1.5} \tag{35}$$

where D_{*k} is the particle parameter with D_{50} replaced by D_k. Eq. (35) indicates that C_{ek} varies more nearly with the inverse of D_k than directly with it, as indicated by (9). Fig. 18(b) shows individual values of the coefficient B in (35) determined as outlined above for the same data as in Fig. 18(a). The plot demonstrates no trend in the values of B that can be related to D. The plot shows considerable scatter, which is unavoidable and that the discusser attributes to the arbitrary locations of the sampling verticals with regard to dune geometry and to the smallness of the suspended sediment concentration samples available for sieving.

The average of all the B values shown on Fig. 18(b) is 0.32. However, Garcia (1995) has provided a compilation of about 200 measurements from deep rivers that can be used to determine a better value for this proportionality constant; the data consist of longitudinal water surface slope, cross-section average suspended sediment concentrations and sieve size analyses, the corresponding average hydraulic parameters, and bed sediment sieve analyses from composites collected uniformly distributed across the measuring section. These data indicate that $B = 0.184$. Furthermore, if (35) is applied to the Scott and Stephens (1966) data using this value for B for subdividing each of the four one ϕ intervals of the bed material into 5 increments (k ranges from 1 to 20), the average value of the ratios of the sums of the C_{ek} to the corresponding values obtained from applying (33) for the appropriate D_{50} and hydraulic conditions is 1.06. Thus, in addition to being uniformly applicable across the sand particle size range, for the Scott and Stephens (1966) data, (35) also yields aggregate values differing only slightly from those of the original van Rijn (1984) expression. The use of (35) is therefore recommended as opposed to (9) when the bed material is to be subdivided into fractions for computing suspended sediment transport.

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Closure by Hong-Yuan Lee,[6] Member, ASCE, Hui-Ming Hsieh,[7] Jinn-Chuang Yang,[8] Member, ASCE, and Chih Ted Yang,[9] Fellow, ASCE

The writers would like to thank Mr. Bennett for the helpful comments. We agree that it is debatable to directly apply van Rijn's (1984) equation to the case of nonuniform sediment. The parameters must be calibrated. However, because of the lack of good quality data, the calibration work is very difficult to perform. In addition, Mr. Bennett has made some comments on the values of the parameters that are very helpful in applying van Rijn's equation. Mr. Bennett also raises the interesting point that these parameters vary with the existence of bed form and flow depth. Therefore, we would like to suggest that van Rijn's equation is, at best, an approximation, and that further investigations need to be conducted.

EFFECT OF SHAPE ON UNIFORM FLOW THROUGH SMOOTH RECTANGULAR OPEN CHANNELS[a]

Discussion by A. K. Kazemipour[2] and C. J. Apelt[3]

The author has described a procedure he followed to derive an empirical expression for the shape parameter, $\psi_p(P/R)$, which enables the coefficient of friction for flow in smooth rectangular channels to be calculated from the coefficient of friction for smooth circular pipes as given by the Karman-Prandtl equation. The latter is his equation (5) in which f' designates the Darcy Weisbach coefficient for smooth circular pipes. His expression for ψ_p, which is the inverse of ψ_K used by Kazemipour and Apelt (1979), is given by (10)–(13). The coefficient for the channel, f, is calculated as $f = f'/\psi_p$.

As pointed out by the author, the shape parameter, ψ_p, is formally related to that developed by Kazemipour and Apelt (1979), designated here as ψ_K, by $\psi_K \equiv 1/\psi_p$. However, ψ_K is obtained by combining two functions, ψ_1 and ψ_2, to give $\psi_K = \psi_1/\psi_2$, where $\psi_1 = (P/b)^{1/2}$. An important benefit from the introduction of the function, ψ_1, is that the values of ψ_2 calculated from experimental data for the coefficient of friction for rectangular channels flow a more clearly defined functional relationship with B/y and with less scatter than is the case for the relationship between (ψ_2/ψ_1) and B/y. In his earlier analysis

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[a]July 1997, Vol. 123, No. 7, by N. Narayana Pillai (Technical Note 9219).

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J. Hydraul. Eng. 1999.125:207-207.

(Pillai 1970a), the author was able to achieve a functional relationship between his single function ψ_p and P/R only after averaging up to 12 individual experimental data points to produce each plotted point for the curve. However, in this recent approach, the author has used the curve for ψ_2 in Fig. 1, developed by the discussers, to calculate the functional relationship between ψ_p and P/R, which is shown in Fig. 2. Because it is derived directly from the well-defined curve for ψ_2, this relationship between ψ_p and P/R is now well defined. However, it differs in varying amounts from the corresponding curve developed by the author directly from experimental data in Pillai (1970a).

The use of a single-shape parameter for smooth rectangular channels, as proposed by the author, does introduce some inconvenience when the parameter is represented by algebraic expressions such as (10)–(13). Nevertheless, the discussers continue to prefer the use of the two functions, ψ_1 and ψ_2, because it is this that gives generality to their approach. The function, ψ_1, is specific to the particular shape of the cross section whereas, as shown in Kazemipour and Apelt (1979), the parameter, ψ_2, can be used for triangular and trapezoidal as well as for rectangular cross sections. Thus, the use of the two functions allows the coefficient of friction for flow in smooth rectangular, triangular, and trapezoidal cross sections to be calculated from the coefficient for smooth circular pipes. In contrast, the single function proposed by the author is applicable only to smooth rectangular channels.

Furthermore, from the results of experiments in smooth semicircular channels reported in Kazemipour and Apelt (1981), it was shown that the use of the two functions, ψ_1 and ψ_2, with ψ_1 defined as $(P/D)^{1/2}$, gave a curve for $\psi_2(D/y_D)$, which could be transformed to that for $\psi_2(B/y)$, as given in Fig. 1, by two linear parallel shifts. In these expressions, D = the diameter of the semicircle; and y_D is defined as A/D. This result extends the generality of the function ψ_2 even further. The discussers recognize that the use of the two function approach would be more convenient if an algebraic expression was fitted to the curve for ψ_2 in Fig. 1. The original data from which the curve was plotted will be reprocessed for this purpose and the outcome will be reported if a satisfactory expression is obtained.

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Closure by N. Narayana Pillai[4]

The writer thanks the discussers, Kazemipour and Apelt, for their keen interest shown in the paper. In his earlier paper (1970a) the writer had used $\psi(P/R)$, which he referred later as ψ_P. The discussers in their paper (1979) developed ψ_K, which happened to be $1/\psi_p$. ψ_K was expressed as ψ_1/ψ_2, the ratio of two nondimensional quantities that are not independent.

The writer derived Fig. 2 from the data of Fig. 1, which is a summary of experimental results to show that $\psi_p(P/R)$ will preserve all of the information from Fig. 1 without any reference to the parameter $\psi_1 = \sqrt{(P/b)}$, and the curve $f'/f = \psi(P/R)$ can be directly used.

For smooth rectangular channels, the writer gives $f'/f = \psi_{Pr}(P/R)$, whereas the discussers give $f'/f = 1/\psi_{Kr}(P/b, B/y)$. However, P/b and B/y are not independent nondimensional quantities and hence one of them is redundant.

For circular channels, the writer gives $f'/f = \psi_{Pc}(P/R)$, whereas the discussers give $f'/f = 1/\psi_{Kc}(P/D, D/y_D)$. However, P/D and D/y_D are not independent nondimensional quantities and hence one of them is redundant.

The writer is of the opinion that with the valuable additional experimental data collected by the discussers separate well-defined curves for $f'/f = \psi(P/R)$ can be obtained for use in rectangular, circular, and channels of other shapes, and their equations can be developed. It may be observed that P/R will have a constant value of 8 for 90° triangular channels and will give $f'/f = \psi(P/R)$, an average value of nearly 0.93, as reported by R. W. Powell in earlier communications.

Errata. The following correction should be made in the original paper:

Page 656, Eq. (6) should read $Q = AV = A(1/N)R^{2/3}S^{1/2} = (A/N)(A/P)^{2/3}S^{1/2} = (A^{4/3}/N)\cdot(A/p^2)^{1/3}S^{1/2}$.

Irrotational Flow and Real Fluid Effects under Planar Sluice Gates[a]

Discussion by Jürg Speerli[2] and Willi H. Hager,[3] Fellow, ASCE

As noted by the author, the flow across a planar sluice gate is a classical problem of hydraulics. Compared with the inverse arrangement of a thin-crested weir, relatively few studies on gate flow are available, and there seems to be no systematic work during the last 20 years.

The purpose of this discussion is threefold:

1. Scale effects
2. Addition of data for the discharge coefficient
3. Spatial flow features

SCALE EFFECTS

Fundamentally, the coefficient of contraction is a quantity difficult to assess. A typical figure that shows the current state is Fig. 1. The author has given some of the reasons for this difficulty, others are highlighted below. From Fig. 14, the energy losses are seen to increase with the absolute gate opening a[m]. For $a \geq 50$ mm, say, the energy loss across a gate is practically zero, and the energy equation gives

$$E = y_0 + \frac{q^2}{2gy_0^2} = C_c a + \frac{q^2}{2g(C_c a)^2} \tag{45}$$

The discharge can be expressed with a modified discharge coefficient C_d^* as

$$q = C_d^* a(2gy_0)1/2 \tag{46}$$

With observed quantities q, a, and y_0, the coefficient C_d^* can be determined. Note that a should be accurately evaluated because of its linear dependence on C_d^*. We use steel elements of precalibrated height glided below the gate, then lower the

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J. Hydraul. Eng. 1999.125:207-207.

gate and move the elements away after the gate is fixed. The height, a, is then correct to at least ± 0.2 mm.

The contraction coefficient C_c is indirectly determined from (45) where eliminating q with (46)

$$C_c = \frac{1}{2} AC_d^{*2} \left[1 + \left(1 + \frac{4}{A2C_d^{*2}} \right)^{1/2} \right] \quad (47)$$

where $A = a/y_0$ = relative gate opening, or approximately by

$$C_c/C_d^* = 1 + \frac{1}{2} (AC_d^*) + \frac{1}{8} (AC_d^*)^2 + \ldots \quad (48)$$

Thus, the ratio of coefficients depends significantly on the contracted flow depth relative to the approach flow depth, and the indirect determination of C_c seems to be much more accurate. Note that the contraction coefficient C_c is always equal to or larger than the discharge coefficient C_d^*.

The previous approach depends essentially on the assumption of constant energy head. For a small relative gate opening, a, this assumption is invalidated by scale effects, including:

- Fluid viscosity
- Surface tension
- Relative boundary roughness
- Exact crest geometry

For sharp crested weirs, these effects are negligible if the overflow depth is at least 50–70 mm, the channel has a width of at least 300 mm, water of common quality of about 20°C is involved, the boundary material is smooth, and the crest has a standard geometry. For the gate flow considered here, similar conditions apply for the Froude Similarity law to be applicable. It is interesting to note that the current data have either been collected on models that do not satisfy these conditions or that details of experimentation are not available.

REVIEW OF RECENT DATA FOR COEFFICIENT OF DISCHARGE

Some recent additional data not considered by the author are now reviewed. Garbrecht (1977) related his laboratory and prototype observations to the free discharge equation (46), and determined for $1.2 < y_0/a < 42$ the discharge coefficient as

$$C_d^* = 0.6468 - 0.1641(a/y_0)^{1/2} \quad (49)$$

Smith (1977) stated the following problems with determination of the contraction coefficient:

- Gate is not precisely sharp-edged
- Presence of shock waves that may not readily be detected

Other reasons, such as the presence of the two-vortex system upstream from the gate or effects of scale have been noted previously. Smith was able to demonstrate that downstream head readings are extremely difficult.

Noutsopoulos and Fanariotis (1978) have determined discharge coefficients based on (46). Their plot includes theoretical results of Fangmeier and Strelkoff (1968) and the experimental data of Henry (1950), and their own. These data may be expressed to $\pm 5\%$ as

$$C_d = 0.62 - 0.15(a/y_0)^{1/2} \quad (50)$$

It was also stated that the reason for the discrepancy between computation and experiment is primarily not the development of the boundary layer but three-dimensional effects, in agreement with the author. The location of the vena contracta was $x/y_0 = 1.73$, almost in perfect agreement again.

Nago (1978) has made a significant contribution to gate flows. His data referred to (46) may be expressed with $\pm 2\%$ as

$$C_d = 0.60 \exp(-0.30a/y_0) \quad (51)$$

All data were collected in a lab channel 400 mm wide, and the gate opening was always $a = 60$ mm. Nago provided an immense number of experiments for a great variety of gate geometries.

Cozzo (1978) reexamined the Gentilini (1947) data and proposed for the vertical thin plate gate

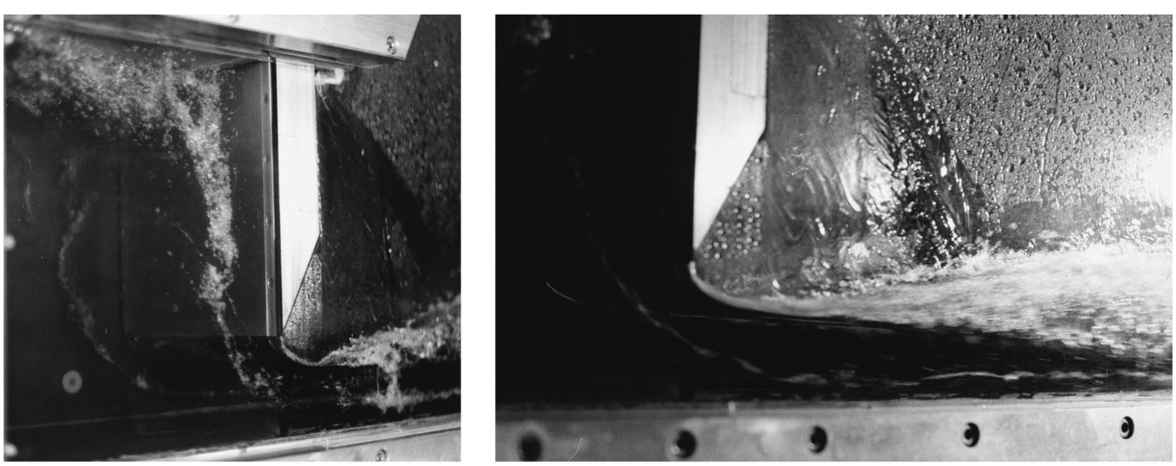
$$C_d = 0.615[1 + 0.300(a/y_0)]^{-1} \quad (52)$$

Note that the first order expansions of (51) and (52) are identical.

Other experimental studies that are worth mentioning and that are not generally known include Franke (1956), Anwar (1964), and Allen and Hamid (1968). In discussing the latter paper, Barr et al. (1969) identify several dimensionless groups of experimental parameters. Given the significance of the vertical gate in hydraulic structures, it can be concluded that the present information on the basic discharge characteristics is incomplete. Some knowledge not referred to by the author is available in reference books such as those by Miller (1994) or Lewin (1995).

THREE-DIMENSIONAL FLOW PATTERNS

The first discusser currently analyzes two-phase flow from a bottom outlet. The main dimensions of the rectangular duct



(a) (b)

FIG. 17. Side Views of Vortex Flows in High-Head Gate Flow: (a) Overall View; (b) Development of Transverse Shockwave

J. Hydraul. Eng. 1999.125:207-207.

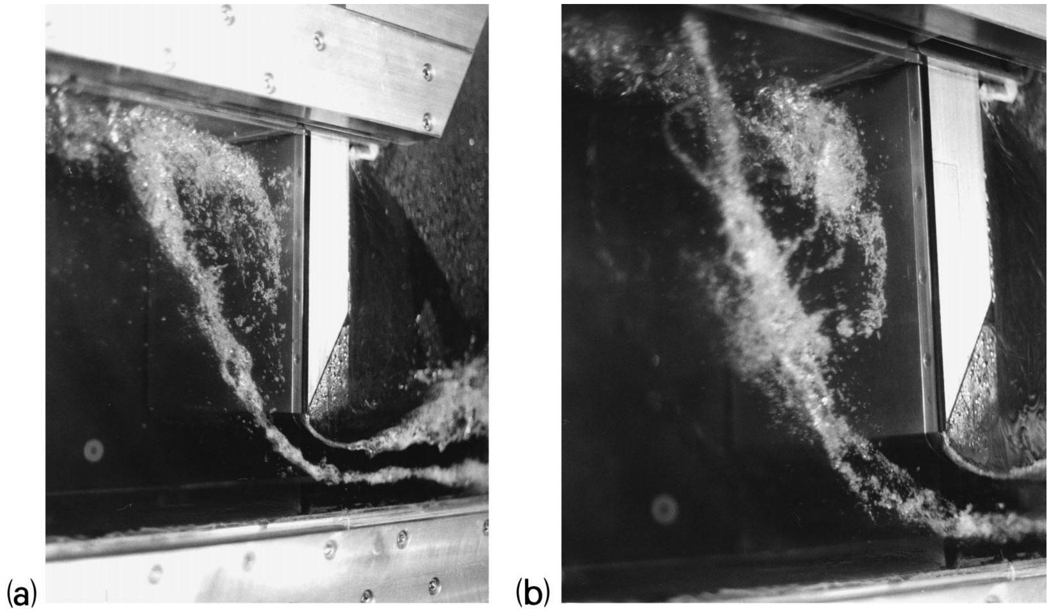


FIG. 18. Details of Corner Vortices: (a) Air Transport Mechanism to Tailwater Reach; (b) Spiral Secondary Cells Wound Up on Primary Vortex

are width 300 mm, approach height 300 mm, and length of outlet tunnel 21 m. Photographs are presented that refer to the flow configuration shown in Fig. 1(b). Air was added upstream from the gate to visualize the vortex structure upstream from the gate.

Fig. 17 illustrates the mechanism of flow as sketched in the upper portion of Fig. 15. All photographs were taken with shutter speeds of either 1/4,000 or 1/8,000 s. The two-vortex system that is suspended on the top boundary is shown in Fig. 17(a). Fig. 17(b) depicts the line vortex on the front side and illustrates the reason for the lateral shock waves that are typical in gate flow. The height of shock is typically of the order of the contracted flow depth.

The intensity of vortex flow along the lateral channel boundaries is noted from Fig. 18(a). This photograph shows a continuous air core, from the upstream top to the downstream region of the gate. Fig. 18(b) shows a closer look to the main vortex, around which spiral secondary cells are suspended. The latter result from the significant corner vorticity, which is generated by the gate close to the top of the approach channel.

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Discussion by M. Grant Webby[4]

The author has carried out a very comprehensive review of previous experimental and theoretical studies of the classical sluice gate problem, including studies not available in the English speaking literature. He comments that ''while most features of the sluice gate flow seem to be predicted accurately by mathematical models based on potential flow theory, there remain persistent discrepancies between the theoretical and experimental values of the contraction coefficient C_c.'' The experimental values generally exceed the theoretical values by 5–10% although experimental values of the discharge coefficient C_d, based on measurements of discharge, show better agreement with theory. The author examines a range of possible explanations for the discrepancies between experiment and theory and concludes that they can best be explained by energy losses induced by the formation of upstream vortices in the corners between the sluice gate and the side walls.

The author points out that the sluice gate is of significant practical importance as a flow control and metering device. Accurate knowledge of the discharge characteristics of a sluice gate used for metering purposes is therefore essential. In practice, sluice gate ratings are often obtained from small scale physical hydraulic model studies with evaluation of the discharge coefficient C_d from measurements of discharge or, alternatively, evaluation of the contraction coefficient C_c from similar measurements using the author's inviscid flow equation (25*a*). In the discusser's experience, such ratings are often not reliable when checked against prototype discharge measurements.

The following discussion compares a recent solution of the radial sluice gate problem with the author's solution of the

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J. Hydraul. Eng. 1999.125:207-207.

planar sluice gate problem. The discussion then examines the accuracy of this solution of the radial sluice gate problem in light of a limited amount of evidence regarding the discharge characteristics of two prototype radial sluice gate structures.

Fig. 19 defines the radial sluice gate problem using the same notation adopted by the author for the planar sluice gate case. The horizontal bed is chosen as the reference datum for the specific energy E. The radial gate geometry is defined by the gate opening a, the gate radius R, the height h of the gate hinge point above the bed, and the gate lip angle θ. However, only three of these four gate parameters are independent as they are related by the equation

$$h = a + R\cos\theta \qquad (53)$$

Isaacs and Allen (1994) used the boundary integral element method as described by Cheng et al. (1981) to carry out a large number of numerical calculations over a wide range of values for the independent variables θ, a/E, and h/E for the radial sluice gate problem. They confirmed the results of some

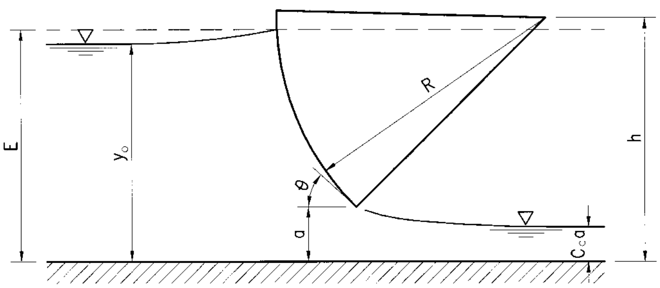

FIG. 19. Geometry of Flow under Radial Gate

of their calculations against a previous finite-element solution by Isaacs (1977) and then analyzed all the results to obtain a simple algebraic equation for the contraction coefficient C_c of the form

$$C_c = \alpha + \beta\cot\theta \qquad (54)$$

in which

$$\alpha = 0.615 - 0.14(a/E) \qquad (55a)$$

$$\beta = 0.14 - 0.10(a/E) \qquad (55b)$$

Additional check calculations showed C_c values predicted by (54), (55a), and (55b) to lie within 0.005 of those calculated by the boundary integral element method in the range 30° < θ < 90° and 0.2 < a/E < 0.6.

The C_c values predicted by (54), (55a), and (55b) for values of θ = 45°, 60°, and 90° are compared in Fig. 20 with the values predicted by the author's inverse type potential flow solution. The two sets of values generally agree within 1.5%.

Values of C_c predicted by (54), (55a), and (55b) were then compared with experimental C_c values inferred from some limited field measurement data for two prototype radial sluice gate

TABLE 4. Radial Gate Geometry of Prototype Sluice Structures

Structure (1)	Number of gates (2)	B (m) (3)	h (m) (4)	R (m) (5)
Arapuni Spillway	4	7	5.3	6.5
Tekapo Canal Inlet Structure	1	13.41	3.505	4.572

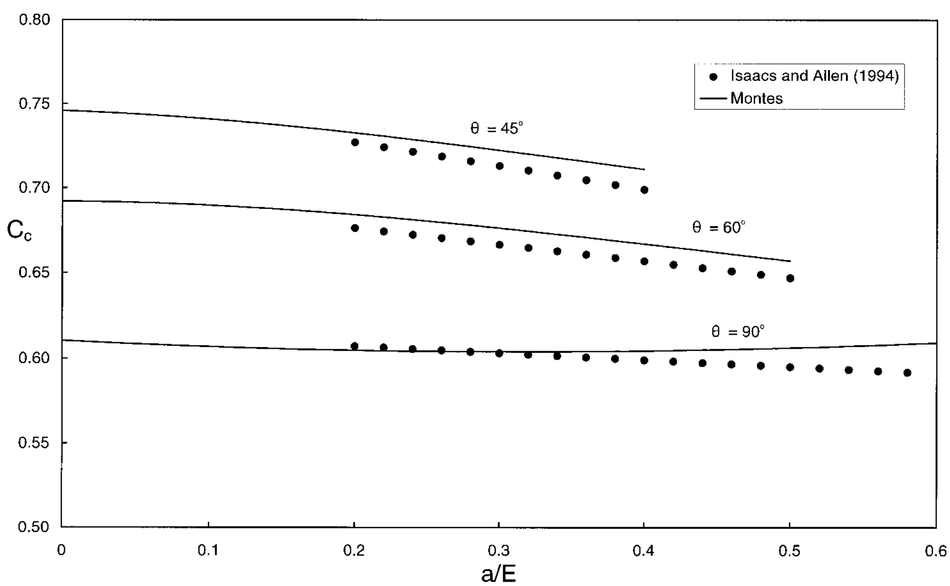

FIG. 20. Comparison of Contraction Coefficient C_c Values Predicted by Potential Flow Solution for Planar Sluice Gates and Isaac and Allen's (1994) Boundary Integral Element Solution for Radial Sluice Gates

TABLE 5. Measured and Calculated Prototype Sluice Gate Structure Data

Structure (1)	E (m) (2)	Q (m^3/s) (3)	a (m) (4)	a/E (5)	C_c Isaac and Allen (1994) (6)	C_c experimental (7)	Difference (%) (8)
Arapuni	3.43	141.5	1.0	0.292	0.701	0.689	−1.7
	3.93	151.6	1.0	0.254	0.706	0.678	−4.0
	3.52	239.6	2.0	0.568	0.641	0.648	+1.1
Tekapo	3.513	112.6	1.959	0.558	0.623	0.645	+3.5
	3.755	97.02	1.453	0.387	0.650	0.675	+3.8
	3.736	81.45	1.181	0.316	0.666	0.678	+1.8
	3.934	64.22	0.816	0.207	0.693	0.724	+4.5

J. Hydraul. Eng. 1999.125:207-207.

structures. It is not practical in such circumstances to measure the flow depth downstream of a gate and obtain the contraction coefficient directly, therefore, the experimental C_c values were back calculated from measurements of discharge using the author's inviscid flow equation (25*a*).

The radial gate geometry of the two prototype sluice structures is summarized in Table 4. The field measurement data and the theoretical and experimental values of C_c are given in Table 5. The discharge measurements were obtained in the case of both structures by current meter gauging in a formed channel during periods of steady flow. The water level measurements were recorded using permanent float-operated water level recorder installations. The vertical gate openings were measured using calibrated gauges attached to the radial gate arms.

The experimental values of C_c in Table 5 for the two structures show different trends when compared with the corresponding theoretical values. For the Tekapo Canal Inlet Structure, the experimental C_c values are consistently higher than the theoretical values, the differences being in the range of from 1.8 to 4.5%. For the Arapuni Spillway Structure, two of the three experimental C_c values are in fact less than the theoretical values, whereas the third experimental C_c value is marginally greater.

Thus, only the Tekapo data support the author's contention that significant energy losses occur between the upstream and downstream sides of a sluice gate structure. However, this support is fairly tenuous because the discrepancies between the experimental and theoretical C_c values are less than the estimated accuracy of the discharge measurements of 5–6%. The Arapuni data on balance would imply an energy gain across the sluice gate structure, which is of course theoretically impossible.

In the discusser's opinion, much more prototype data needs to be collected before a definitive statement can be made regarding the significance of energy losses across a prototype sluice gate structure (as opposed to a model structure) due to the formation of upstream corner vortices. Until such data are collated and analyzed, the discusser suggests, for practical purposes, continuing to assume negligible energy losses across a prototype sluice gate structure and deriving the discharge characteristics of the gate from a solution such as Isaac and Allen's (1994).

ACKNOWLEDGMENTS

The permission of the Electricity Corporation of New Zealand to use the gauging data obtained from the Tekapo Canal Inlet Structure and the Arapuni Spillway is gratefully acknowledged.

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Closure by J. S. Montes[5]

The writer thanks the discussers, Speerli and Hager and Webby for their comments and for the additional references that they have presented regarding the coefficient of discharge and contraction under vertical and inclined gates. Much appreciated are the photographs by Speerli and Hager of the vertical vortex forming just above a vertical gate. The visualization of these phenomena is not easy and the strong vortex action shown in these photos may come as somewhat of a surprise to many engineers.

Although the influence of the vortex field on the gate flow is a subject under study at present at the Department of Engineering of the University of Tasmania, some preliminary features of the vortex phenomenon can be advanced. Perhaps the most important is the fact that the existence of the vortex depends on the angle of the gate. In general, when the gate has angles with respect to the horizontal sensibly below 90° the vortex will not form. On the contrary, when the gate structure leans forward at the top ($\theta > 90°$), the vortex is most intense. The Tainter gate described by Webby falls in the earlier category, because its mean angle of inclination with respect to the horizontal is well below 90°. Thus, one may not expect vortices to form upstream of Tainter gates and few such vortices can be observed in the field.

Both discussers comment directly or indirectly on the phenomenon of energy loss through the gate. The experimental evidence is clear that perceptible energy losses occur in flow under vertical gates, following from the experiments of Fawer, Valentin, and Rajaratnam. This is why the method of computing C_c from the measured values of C_d, as proposed by Smith (1977) and adopted in their discussion by Speerli and Hager, cannot be an accurate procedure for gates whose angles of inclination equals or exceeds 90°, because the assumption of constant energy is made implicitly in this derivation.

When there is a certain amount of energy loss between the upstream and downstream sections of the gate, the calculated value of C_c must be at variance with values deduced from (47) or (48). If ε is the percentage of upstream energy lost in the flow below the gate, then one may write

$$(1 - \varepsilon)\left(y_0 + \frac{q^2}{2gy_0}\right) = C_c a + \frac{q^2}{2g(C_c a)^2}$$

The discharge q can be defined by means of (46) using Garbrecht's empirical expression for flow under vertical gates, as quoted by Speerli and Hager. Notice that this expression for the discharge coefficient obtained by a simple correlation between upstream head and discharge is independent of the existence of energy losses. The comparison depicted in Fig. 21 indicates that the potential flow C_c values are obtained within 0.5% when the energy loss is assumed negligible ($\varepsilon = 0$). But the measured values of C_c are different. Now the comparison of the computed values including energy loss with the trend of experiments of Fawer, Smetana, and Gibson as taken from Fig. 12(a) of the original paper, shows that the likely amount of energy loss is about 3% at low values of a/y_0 and more than double that amount when a/y_0 nears its maximum value. This is in broad agreement with the original estimates. On the other hand, the coefficient of contraction for inclined gates as computed from Gentilini's experiments shows good agreement with the irrotational flow theory [Fig. 12(b) of the original paper], which would lead to the conclusion that energy losses in that case are negligible. As previously commented, no vortex could be observed when the plane gate was inclined at 60° or 45°, or in the case of a Tainter gate with similar lip inclination.

The work of Anwar (1964), whose reference was contributed by Speerli and Hager, provides some additional experimental evidence on flow under inclined gates. Fig. 22 shows a comparison of the computed gate pressure using the inverse potential flow method of the article and the experiments of Anwar conducted at the HRS, Wallingford. The difference between the calculated and experimental values is small and of the same order as the values computed by Anwar's conformal transformation method used in Kirchoff's classical theory.

Webby presents additional data based on field experiments

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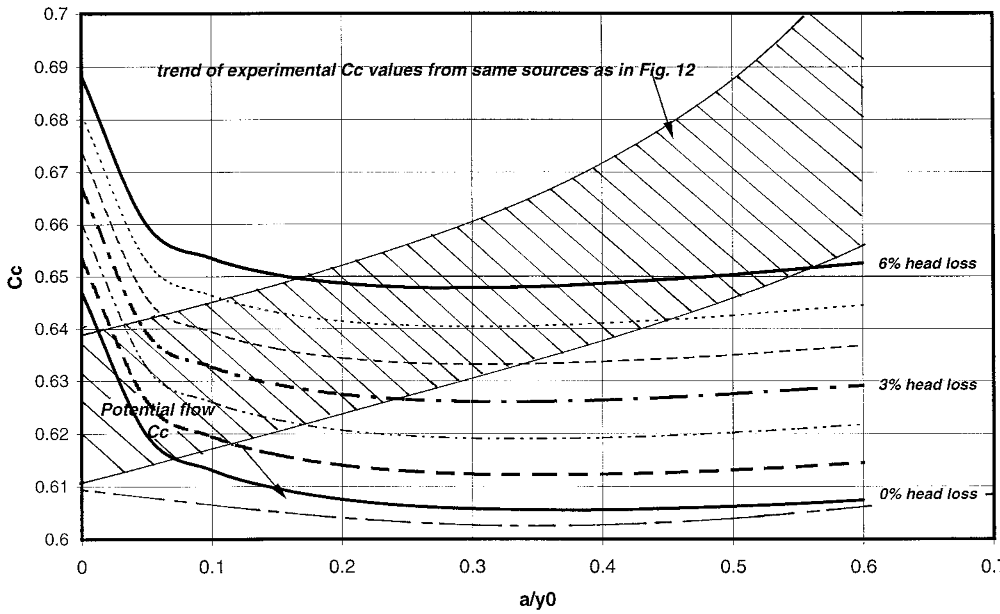

FIG. 21. Coefficient of Contraction C_c Computed from Garbrecht's Expression for C_d Assuming Various Values of Energy Loss (When the Energy Loss Is Nil, Calculated Value Is Very Close to Potential Flow Value)

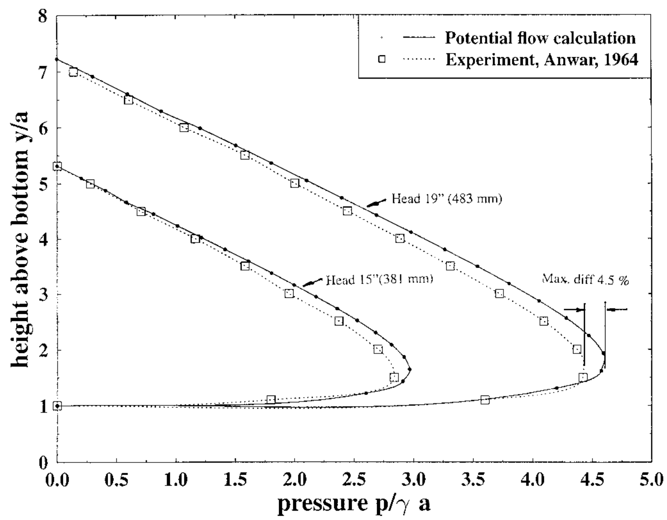

FIG. 22. Comparison of Pressures along a 50° Inclined Gate, as Computed from Potential Flow Numerical Scheme and Experimental Data of Anwar (1964) (Maximum Difference Is of the Order of 4%)

in New Zealand. Although it is mentioned that the discharges were estimated using current meters, there is no mention of the methods used in measuring the coefficient of contraction. It is assumed that the values of C_c quoted by Webby are indeed found from direct measurement of the downstream depths and not derived from the discharge equation. The precise measurement of the depth at the section of maximum contraction under highly turbulent flow conditions is by no means an easy operation in the field, and further details in this regard would have been of value. Given these practical difficulties, the discrepancy between the values of C_c from the potential flow calculations of Allen and Isaacs (1994) and the experimental values is quite small.

The similarity of the experimental and theoretical values of C_c for radial gates is used by Webby as an indication that the idea of energy losses for flow under a gate is still in contention. However, as indicated in the first part of this closure, the energy losses are connected with the vortex field upstream, which practically disappears when the inclination of the gate descends below 60°, precisely the case of the New Zealand experiments. One would expect then a good agreement between the experimental coefficient and those determined from the potential flow computation.

It is gratifying to see the reasonable agreement between the potential flow computation of Allen and Isaacs and that of the writer. This had been the case with several other methods quoted in the original paper, showing that the small differences that exist can be attributed to slightly different boundary conditions; for example, the length of the channel allowed before parallel flow is found both upstream and downstream from the gate. The mechanism of energy losses for the vertical gates still remains to be fully explored, but it is indicative of the plausibility of the basic argument that only when there is an appreciable vortex field upstream of the gate are there significant differences between the experimental and theoretical values of C_c.

CHARACTERIZATION OF STREAM MEANDERS FOR STREAM RESTORATION[a]

Discussion by S. V. Chitale[3]

Comments made by the authors about the Leopold-Wolman equations not being applicable to the problem of channel restoration design undertaken for the 18 streams in Maryland appear to be not wholly justified. These formulas were evolved in 1960, when information was scant on the factors governing the river morphology. The Leopold-Wolman equation for the meander wave length λ, amplitude A, and radius of curvature R_c in terms of the channel width W quoted by the author are now known to be incomplete. It is presently realized (Chitale

[a]June 1997, Vol. 123, No. 6, by Massimo Rinaldi and Peggy A. Johnson (Technical Note 9968).

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J. Hydraul. Eng. 1999.125:207-207.

1970; Chang 1985) that besides W, which scales the meander size, the meander sinuosity P, defining the meander shape, is also another factor controlling the values of λ, A, and R_c. Furthermore, for any specific combination of W and P, the values of λ, A, and R_c are uniquely fixed.

For application of these findings, the relationships of the meander size and shape need to be known. The interrelation between P, A, and W defining the meander size was evolved (Chitale 1970) on basis of data of mostly the Indian rivers as

$$P = 1.145 \left(\frac{A}{W}\right)^{0.134} \quad (7)$$

Error of estimation equal to

$$\sqrt{\frac{\sum (\text{actual value} - \text{estimated value})^2}{\text{number of observations}}} \times \frac{100}{\text{mean of actual values}}$$

for this equation was 18.8%. The range covered by this data was W, 107–9,451 m (350–31,000 ft); D, 1–20 m (3.20–66 ft); $S \times 10^4$, 0.167–43; P, 1–2.25; Q, 142–52,370 m^3/s (5,000–1,848,000 cfs); and d_{50}, 0.1–5.00 mm. Knowing P and W, A can be estimated by (7).

Making use of the well-known characteristics of the meandering sinuosity varying inversely with d_{50}/D, S, and W/D, the relation for P was worked out on the basis of the same Indian data (Chitale 1970) as

$$P = 1.429 \left(\frac{d_{50}}{D}\right)^{-0.077} S_*^{-0.052} \left(\frac{W}{D}\right)^{-0.065} \quad (8a)$$

where $S_* = S \times 10^4$. Error for this equation was 20%. The Indian river data pertained to both the alluvial and incised rivers. For purely alluvial rivers, the coefficient of (8*a*) was recalibrated using the data of mostly United States rivers (Schumm 1969). The equation obtained was

$$P = \left(\frac{d_{50}}{D}\right)^{-0.077} S_*^{-0.052} \left(\frac{W}{D}\right)^{-0.065} \quad (8b)$$

The error for this equation was 7.76%. Knowing d_{50}, D, S, and W, the value of P can be estimated by (8*b*).

Values of λ, A, and R_c depend on both W and P. The meander shape may be described as a sine curve, parabolic curve, circular curve, or sine-generated curve. Field data show that a sine-generated curve approximates the meander shape fairly well (Langbein and Leopold 1966). For such a curve the relations obtained are

$$P = \frac{2.5}{(\lambda/A)^{0.52}} \quad (9)$$

$$P = 0.52 \left(\frac{\lambda}{r_c}\right)^{0.52} \quad (10)$$

With the above relations it is possible first to estimate the regular stable planform of the meander shape; sinuosity P for the given data of W, D, S, and d_{50} by equation (8*b*). The value of A can then be determined by (7). Eqs. (9) and (10) give corresponding values of λ and R_c.

After A, λ, and R_c, are worked out, these values need be compared with the data of the river being considered for channel restoration and adjustments made, if found necessary, on account of any extraneous reason such as natural constraints or through human agency.

Such an attempt is expected to be fruitful, yielding a more rational solution of the river development problem.

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Discussion by R. D. Hey,[4] Member, ASCE

The authors are to be commended for highlighting potential problems associated with the uncritical application of empirical equations for characterizing meander plan form for river restoration schemes. By comparing observed values of meander wavelength (λ, ft), amplitude (A, ft), and radius of curvature (R_c, ft) for small streams in Maryland with those predicted by Leopold and Wolman's (1960), (1)–(3), based on channel width (W, ft)

$$\lambda = 10.9W^{1.01} \quad (1)$$

$$A = 2.7W^{1.1} \quad (2)$$

$$R_c = 2.4W^{1.0} \quad (3)$$

they showed that the predicted values grossly overestimated observed ones. Before applying (1)–(3), or their metric equivalents, for restoration design, they concluded that due allowance would have to be made for any local bias. The reason for the large bias was considered to result from a combination of the lack of regular and well-defined meanders, heavily vegetated banks on small channels, and adjustment induced by urbanization.

Because two variables are required to uniquely determine the plan form of regular meander bends (Hey and Thorne 1975), two equations need to be defined in order to accurately prescribe meander geometry for natural channel design purposes. For irregular meanders additional parameters would be required. The above equations relating meander wavelength, amplitude, and radius of curvature to channel width are typical of the many empirical equations that have been used for this purpose.

Eqs. (1) and (2) for predicting wavelength and amplitude are probably best disregarded, in spite of their popularity, partly because of the difficulty in defining representative average values where bends have an irregular shape, but principally because it is inappropriate to use a straight-line measurement to characterize a meandering channel when the formative processes must occur in-bank.

In contrast, information on radius of curvature and channel width is much more precise and specific to a particular bend. Nevertheless, it should be recognized that this imposes a circular arc form on the meander bend, even though it may correspond more closely to a sine-generated curve. Significantly, (3) is also process based, as Bagnold (1960) showed that flow resistance in meanders is a function of the ratio R_c/W. For ratios less than two, tighter bends, flow resistance increases substantially due to flow separation at the exit from the bend.

Data obtained from unconfined meander bends on the Rivers Wye and Tweed in the United Kingdom, which were in regime, have been used to assess the validity of (3) (Hey 1976). This indicates that it only applies to rivers with arc angles, measured between inflexion points on the approach to, and exit from, each bend of 150° and that R_c/W ratios vary inversely with arc angle. It was also shown that (1) and (2) are specific to arc angles of 150°.

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J. Hydraul. Eng. 1999.125:207-207.

More general equations are available for defining the plan form of unconfined meanders (Hey 1976), which actually relate to the flow processes operating in the river, and these have been successfully used for designing river restoration schemes (Hey 1994). The first is obtained from the definition of sinuosity (p)

$$p = S_v/S \tag{11}$$

where S_v = valley slope, and S = channel slope. Provided the valley slope is fixed, then any change in channel slope through erosion or deposition to a new regime condition will result in a new plan form. However, as there are a multiplicity of plan shapes for a given sinuosity, the exact shape is dependent on defining another meander parameter.

A second general relationship has been established between meander arc length (Z), measured around the bend between inflection points, and channel width that enables the plan form to be uniquely determined (Hey 1976; Hey and Thorne 1986)

$$Z = 6.28W \text{ (ft/m)} \tag{12}$$

This is also equivalent to the general observation that riffles are spaced between five and seven channel widths apart (Leopold et al. 1964; Richards 1976); possibly more closely spaced on steeper channels, which would conform with the step spacing in step-pool channels.

Secondary flows, which strongly influence velocity distributions and zones of surface flow convergence, bed scour, divergence, and bar deposition, are probably responsible for this average spacing. Given that two secondary flow cells are present at high channel forming discharges, with an average size of one-half the width of the river (Bathurst et al. 1977, 1979; Dietrich et al. 1979), then spatial correlation indicates that the velocity pattern will repeat itself at intervals of $4\pi W$ along the river, which gives a meander arc length of $2\pi W$ (Hey 1976).

Evaluation of the Maryland data showed that at many sites the radius of curvature was less than channel width (sites 1, 3, 5, 6, 12, 13, 14, 15, 18) and at others the channel width exceeded the amplitude of the meanders (sites 2, 3, 14), both of which would appear to be anomalous. To check the consistency of the various measures a sketch of two regular meander bends, one wavelength, was made based on the data for site 18. Given the wavelength (15.3 m) and amplitude (5.0 m), the radius of curvature of each bend has to be 4.5 m, and not the tabulated 2.0 m if the bend is a regular circular arc and the radius of curvature was measured between adjacent inflexion points. This produces a meander arc length of 9.1 m, an arc angle of 100°, and a sinuosity of 1.19. The latter corresponds to the observation that sinuosities were generally less than 1.2. Given the observed channel width, 3.9 m, the riffle spacing would still only be 2.33 W; an exceptionally low value. To be compatible with other U.S. and U.K. data, channel width would have to be 1.45 m, giving a R_c/W ratio of 3.1, compared with an observed ratio of 0.51. Similarly for the other sites, R_c and W values are not consistent with λ and A values. While recognizing that these sites had an irregular plan form, whereas the foregoing assumed symmetry, it emphasizes the difficulty of obtaining sensible averages that are internally consistent.

If all the Maryland sites really have a lower R_c value than that predicted by (3), and hence an even lower value than that based on (12), it implies that these Maryland rivers are significantly different from other U.S. and U.K. rivers.

The authors speculate that bank vegetation would have a significant impact on meander geometry and, thereby, could explain these differences. Because the U.K. data contained both tree-lined and nontree-lined banks (Hey 1976), and it was subsequently shown that riffle spacing/meander arc length was not significantly affected by the presence or absence of trees (Hey and Thorne 1986), it follows that bank vegetation does not explain this disparity. This does not mean, however, that bank strength, which can be enhanced by bank vegetation on smaller rivers, is not a determinant of meander geometry. To illustrate this, it is well known that for a given valley slope and bed material calibre there are a multiplicity of channel widths, depths, and slopes that can transmit a specified discharge and load. The actual cross-sectional, longitudinal, and plan form are strongly influenced by bank strength (Hey 1997). Low bank strength produces a wide, shallow, relative straight, and steep channel, whereas rivers with greater bank strength are narrower and deeper with a low gradient and, thereby, more sinuous. Hence, tree-lined channels are likely to be more sinuous and have a smaller arc length than nontree-lined ones, other things being equal.

After visiting some of the field sites with one of the authors it was apparent that the Maryland rivers were not unrestricted free formed meanders and, therefore, represent a different population of rivers. The streams are relatively small and steep and are confined within a narrow flood plain. There was also some evidence of incision, either generally due to urbanization increasing flood run-off or, locally downstream from road culverts, which can cause problems for defining bankfull conditions. Given the limited flood plain width, the geometry of individual bends was strongly influenced by contact with the steep valley side and, because of the small size of the streams, the rooting systems of individual large trees also controlled meander development. Hence, meander plan form was strongly influenced by the location of hard points. The coarse nature of the bed material indicated that in-bank flows were not competent to transport all the size fractions and that these rivers adjust to the largest floods, because there is limited opportunity for the channel to be reshaped during lesser events. This indicates that these rivers are incised stable or unstable channels. The angularity of the bed material attests to the fact that it is infrequently transported. As a consequence, the location of riffles is not hydraulically controlled and is independent of the meander plan form. It would appear that they reflect local downstream changes in transport capacity of the stream as influenced by valley geometry and the flashiness of the flood event. Bar deposits, riffles, were located where stream competence declined. These streams therefore are neither free formed nor in regime. By contrast, the U.K. data were as if they had no lateral restrictions, no local dominating hard points, and in-bank flows were competent to transport all the bed material.

The application of the Maryland equations for river restoration and stabilization also merits consideration. If it is intended to restore free formed meanders, then the equations based on Maryland rivers are unlikely to be appropriate for the reasons outlined above, in which case the more general plan form (4) and (5) would provide the best basis for specifying the required meander geometry. Alternatively, if it was necessary to stabilize rivers with constricted valley bottom land by armoring the bed and incorporating hard points, then empirical Maryland equations would be more appropriate for specifying meander pattern. However, designs would only be successful if the cause of the instability was controlled to ensure that there was no further change in the boundary conditions, that the full length of the unstable reach was restored and that the load transported in the reach, volume and calibre, corresponded to those in the adjacent stable reaches.

This contribution confirms the view of the authors that it is necessary to ensure that empirical equations are not applied out of context and, if possible, their local applicability should be verified. In this context it would appear that these particular Maryland streams are different, because the streams are not free formed and they are probably degrading due to urbanization. If this is the case, then the use of such equations would

J. Hydraul. Eng. 1999.125:207-207.

be inappropriate for designing stable restoration schemes where there was no lateral constriction on meander plan form. Instead the scheme would need to be designed using general equations derived from free formed rivers that are in regime after first confirming their local applicability.

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Discussion by Chester C. Watson,[5] Fellow, ASCE, and David S. Biedenharn,[6] Member, ASCE

The authors are to be complimented for their appropriate recognition of the need to utilize a combination of field reconnaissance and analytical study in developing empirical relationships. Leopold et al. (1964) also presented the results of various authors who related meander amplitude to channel width. For example, (2) in the authors' paper has a coefficient of 2.7 and Leopold et al. (1964) provided similar relationships with coefficients of 2.7, 10.9, and 18.6. Therefore, it is not surprising that data sets from Leopold and Wolman (1960) do not represent the relationships for data collected by the authors. The authors clearly represent methods for quantifying the differences and are to be commended for their efforts.

The discussers are concerned by the heavy reliance on empirical relationships for stream restoration design. Mark Twain (1944), in *Life on the Mississippi*, Chapter 17, alludes to the use of empirical relationships by predicting that, ''. . . the Lower Mississippi will be only a mile and three-quarters long, and Cairo and New Orleans will have joined their streets together . . .'' based on his observed shortening of the Mississippi River by cutoffs. For many of us who use empirical relationships, we should remember Twain's summary of his pointed sarcasm, ''There is something fascinating about science. One gets such wholesale returns of conjecture out of such a trifling investment of fact.''

Recently, Wilcock (1997) made the following comments concerning the dependence on empirical relationships by river restoration practitioners.

> Mistakes are made, particularly when an apparently stable reach is used as a template for the restored channel or when general empirical relations between channel geometry and flow frequency are used as the basis for design. . . . Although the general empirical relations for river channel geometry are real and pose a continuing challenge to those who would predict river behavior, they provide an inappropriate basis for river restoration. The wrong questions are being asked.
>
> A more useful paradigm is that rivers adjust to the water and sediment supplied to them. . . . This approach leads immediately to the cause of channel response: changes in water and sediment supply. . . . When the design goals are clearly defined and based on general physical principles (rather than referenced to a variously and empirically defined equilibrium state), physical, ecological, and management objectives can be considered consistently and quantitatively.

The discussers are in no way criticizing the authors' efforts; however, we do strongly emphasize that stream restoration design basis should be escalated to the fundamental level of developing a balance between supply and transport of water and sediment for a desired level of sediment yield from the upstream watershed and channel system. Continued reliance solely on empirical relationships will mire the profession in the fixed-boundary hydraulics and regime relationships established decades ago.

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Closure by Massimo Rinaldi[7] and Peggy A. Johnson[8]

The writers thank the discussers for their thoughtful contributions. Hey clearly addresses the problem with using the Leopold and Wolman equations (or any other empirical equations) in defining meander geometry. He also points out the benefits of using meander arc length, z, rather than meander wavelength, λ, and provides a semiempirical equation for free-formed meanders. For newly constructed meanders that are not yet stabilized by vegetation, the use of a free-form meander geometry is probably preferable because it is likely to be the most stable form. The regional equations discussed in our paper would be valid for use only if such hard points (vegetation, etc.) were in place. In Hey's data set, however, there were only three data points that were similar in magnitude (bankfull width) to our data set. Therefore, this relationship should be tested using additional data for small channels before using. Chitale has presented additional equations for describing meander planform. Again, these equations are based on data outside the range of the small eastern streams and caution is strongly advised in directly using these results.

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J. Hydraul. Eng. 1999.125:207-207.

The reference made by Watson and Biedenharn to Mark Twain's *Life on the Mississippi* is terrific. This novel is a wonderfully enlightening reflection on river behavior and we highly recommend it to all of those interested in river hydraulics and mechanics. These discussers also point out that it is important in any river modification design that the design be checked to ensure that it can convey the sediment and flow discharge entering at the upstream end. We certainly agree with this.

Finally, the most important message here is to be very cautious in applying meander planform equations in design. With the exception of Hey's study, the equations are empirical in nature and, thus, are not applicable outside of the magnitude and physiographic ranges of the data to calibrate those equations.

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