Discharge Formulas of Crump-De Gruyter Gate-Weir for Computer Simulation

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Abstract: One of the problems of interest to professionals in the field of irrigation and drainage is the computer simulation of discharge or level control structures. Particularly troublesome are structures that display a marked change of behavior when the downstream water level exceeds a certain limit. The Crump-de Gruyter gate displays several such changes of behavior. Not only does it exhibit a transition from free to drowned flow when the downstream water level rises, it can also go from weir to gate flow. A series of experiments in a laboratory flume provided the basic data to test a simple mathematical model of this structure. The model assumes the structure is located between two reaches with sub-critical flow in the upstream and downstream reach.

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Introduction

Gates with weir-like sills (Fig. 1) are used as discharge (or level) control structures within water management systems. The calculation of the discharge based on gate position, upstream water level, and downstream water level is difficult. Even if we assume subcritical flow in the upstream and downstream reaches, there are still no less than five different flow states that may occur and for each state a separate discharge formula is needed. Moreover, the discharge dependent transition points between these formulas need to be determined. The possible flow states for a gate with a weir-like sill are (cf. Fig. 1)

- A Weir flow (the gate does not touch the water surface)
  A1 Free weir flow, i.e., critical depth over the weir, if a hydraulic jump occurs then it lies downstream of the weir, (small) downstream disturbances do not propagate upstream.
  A2 Drowned weir flow, i.e., the water depth over the weir is greater than the critical depth, downstream disturbances can propagate upstream.
- B Gate flow (the gate opening is less than the critical depth for the discharge)
  B1 Free gate flow, the hydraulic jump occurs downstream of the weir, (small) downstream disturbances do not propagate upstream.
  B2 Drowned gate flow, a drowned hydraulic jump on the weir touching the gate, downstream disturbances can propagate upstream.
- C Transitional flow, the gate touches the water surface, but the gate opening exceeds the critical depth for the discharge.

Traditionally a designer would establish in advance which flow state would best serve his purpose and then design the structure in such a way that only this flow state would occur under normal operating conditions. With the advance of computer simulations we can evaluate the hydraulic performance of water systems under different operational conditions: We can vary the operational strategy, take into account the usual wear and tear, and subject the system to external disturbances. As a result, unplanned transitions between flow conditions may occur as the system leaves the range of operating conditions considered during its design. Unless the simulation can accurately model these transitions its usefulness as a planning tool will be severely limited. Therefore we need a mathematical model suitable for inclusion in a simulation package.

The derivation of a mathematical model can be split into two subproblems. Namely, determination of the flow state and derivation of a relation between discharge, upstream water level, downstream water level, and gate position for that flow state.

As a first step towards this goal this paper studies the behavior of a laboratory model of a Crump-de Gruyter gate (de Gruijter 1925a,b,c, 1926, 1927a,b; Vlugter 1927, 1932; Romijn 1938; Bos 1990, p. 286). This is a gated weir with a rounded upstream edge on its gate leaf. The rounding is intended to eliminate contraction effects at the downstream edge of the gate.

A series of experiments in a laboratory flume provided the basic data that will be used to test the mathematical model. Some of the experiments and some of our results were originally reported by Spaan (1994). Additional experiments were performed by de Graaff (1998).

The remainder of this paper has three main themes: Construction of an algorithm to determine flow state and discharge, comparison of the predictions of this algorithm with our experimental results, and finally the presentation of our conclusions.

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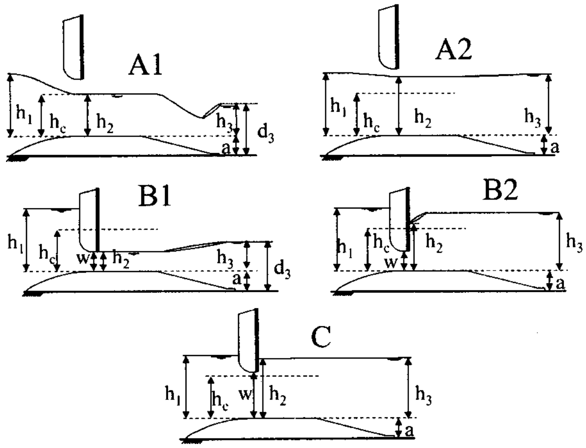

Fig. 1. Sketches of different flow states

Theoretical Flow State and Discharge

In the following analysis, we use the standard expressions for energy and momentum at a cross section. We do not include correction factors for nonuniform velocities in the velocity term of these formulas because, without velocity field measurements, their effects cannot be properly separated from the effects of energy losses in the system. We assume that the rounded shape of the gate allows us to take the contraction coefficient of the gate equal to one without significantly reducing the accuracy of the resulting formula. In our analysis we use three numbered cross sections. They are located as shown in Fig. 2: Cross Section 1 just before the contraction at the upstream end of the structure, Cross Section 2 immediately downstream of the gate, and Cross Section 3 just after of the expansion at the downstream end of the structure. Note that all cross sections are rectangular.

The following notation will be used. We use Q for the discharge, a for the height of the weir, w for the gate opening, and g for the gravitational acceleration, which we take to be 9.81 m/s^2. Some quantities carry as a subscript the number of the corresponding cross section. The width of a cross section will be denoted by B, the water depth by d, the water level relative to the crest of the weir by h. The energy loss between two sections i and j will be denoted by ΔE_{ij}. Note that the formulas used assume that the flow is unchanging in time (steady flow).

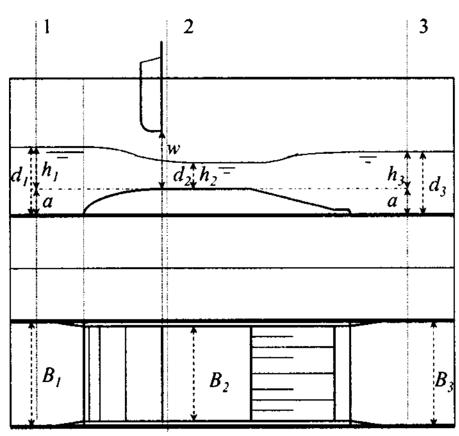

Fig. 2. Location of cross sections

Initial $Q-h$ Relationship

We base our initial $Q-h$ relationship [Eq. (1)] on the energy balance between Cross Section 1 upstream of the structure and Cross Section 2 just past the gate. The effect of the gate is included through restriction of the flow cross section to the gate opening.

$$h_1+\frac{Q^2}{2gB_1^2d_1^2}=h_2+\frac{Q^2}{2gB_2^2\min(w,h_2)^2}+\Delta E_{12} \tag{1}$$

As in Henderson (1966), in Eq. (1) the depth over the weir is allowed to differ from the gate opening to allow for a drowned gate. Note that the use of the hydrostatic pressure head at Cross Section 2 is not wholly justified. However, the remainder of this paper will show that the resulting formulas can be quite useful.

From Eq. (1) we derive a form more suitable for discharge predictions [Eq. (2a)] by moving the discharge term on the left-hand side to the right-hand side and assuming that the energy loss in the contraction can be approximated by a coefficient times the difference in the velocity head terms and included in a factor C^2 [defined in Eq. (2b)].

$$2gB_2^2C^2\min(w,h_2)^2(h_1-h_2)=\left(1-\frac{B_2^2\min(w,h_2)^2}{B_1^2d_1^2}\right)Q^2 \tag{2a}$$

$$\frac{1}{C^2}=1+\frac{2g\Delta E_{12}}{(V_2^2-V_1^2)} \tag{2b}$$

Such an assumption is not unreasonable, given the use of similar approximations in literature (Chaudhry 1993; Henderson 1966, etc.). This provides us with an initial $Q-h$ relationship, but leaves open the problem of determining the water depth over the weir. Moreover, it contains an unknown coefficient C. This coefficient represents the energy loss for a specific structure, so its value is best determined experimentally. We will deal with the problem of water depth below. The details of the choice of C will be discussed later, together with our algorithm.

Substitute for Depth on Weir

For the formula implied by Eq. (2) to be useful we need an approximation for h_2. Given the different situations for weir and gate, we need separate estimates for this level. Our estimates will be based on the expressions for momentum at a given cross section.

Weir

From Fig. 3, it is clear that the head loss over the drowned weir is very small. To distinguish between different discharges for a given downstream water level we need to know the head loss with an accuracy that is not likely to be attainable in the field. For example, in our laboratory setup the difference between 27 and 35 L/s is less than 0.5 mm (Fig. 3). It would seem that an accurate estimate of the water level over the weir is not the central problem. Therefore, in the case of the drowned weir we use the standard approximation for expansions, i.e., we take the force on the sides and bottom of the expansion equal to the static force that a water level equal to the downstream water level would exert. This results in the momentum balance given in Eq. (3a)

$$\frac{B_2}{2}h_2^2+\frac{Q^2}{gB_2h_2}=\frac{B_2}{2}h_3^2+\frac{Q^2}{gB_3d_3} \tag{3a}$$

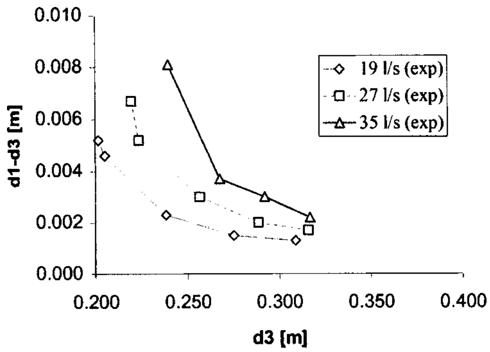

Fig. 3. Head loss versus downstream water level for fixed discharge for transition from free to drowned weir flow

For the free weir we use the critical depth corresponding to the discharge, given by Eq. (3*b*) to provide the water level over the weir

$$h_2 = h_c = \sqrt[3]{\frac{Q^2}{gB_2^2}} \tag{3b}$$

Gate

For the gate, we calculated the force on the weir from our experimental data and found an alternative to Eq. (3*a*) that results in a better model of the free gate to drowned gate transition (cf. the section on comparison of theory and experiment). We will now describe this alternative.

We combine the small expansion due to the widening of the canal and the straight drop at the end of the weir and approximate the forces on the walls and the toe of the weir by the static pressure corresponding to the downstream water level [Eq. (4)]

$$F_e = \rho g \frac{B_3 d_3^2}{2} - \rho g \frac{B_2\left(d_3 - \frac{a}{5}\right)^2}{2} \tag{4}$$

The calculation of the force on the downstream end of the weir is based on the following approximation of the water level. Assume the water level follows a quadratic curve running from the water level h_2 just past the gate (at Cross Section 2) to the level h_3 at the vertical drop at the end of the weir (Fig. 4). The curve has a horizontal tangent at the latter point.

This results in a horizontal force component as described by Eq. (5) on the slope of the weir. When combined with Eq. (4) this provides us with Eq. (6*a*) for h_2 with ζ as specified in Eq. (6*b*)

$$F_s = \rho B_2 g \int_{x=3/4}^{7/4-1/5} a h_2 + a^2\left(x - \frac{3}{4}\right) + a(h_3 - h_2)\left[2\left(\frac{4x}{7}\right) - \left(\frac{4x}{7}\right)^2\right] dx \tag{5}$$

$$h_3^2 - h_2^2 + 2a\zeta(h_3 - h_2) = \frac{2Q^2}{gB_2^2 w}\left(1 - \frac{B_2 w}{B_3 d_3}\right) \tag{6a}$$

$$\zeta = \frac{4}{5}\left(1 - \frac{3{,}179}{3{,}675}\right) \approx 0.108 \tag{6b}$$

Remark: $\zeta = 0$ corresponds to the conventional approach that we used for the weir (force based on downstream water level) whereas $\zeta = 1$ corresponds to a force on the weir based on the water level at Cross Section 2. A discussion of the effects of different choices of ζ can be found in the section on comparison of theory and experiment.

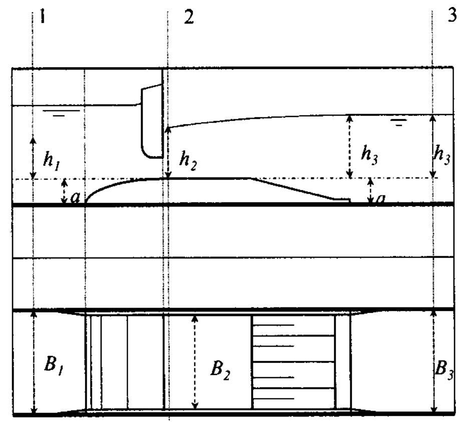

Fig. 4. Sketch of suggested water surface curve

Next we will use our results to derive a formula for h_2 that does not contain Q and as such can be used to determine the discharge.

Algorithm to Determine Flow State and Discharge

The previous sections have provided us with, on the one hand, a discharge formula in terms of the upstream depth and the water level over the weir, and on the other hand formulas for the water level over the weir and the downstream water level and the discharge.

Please note that Eq. (2), the starting point of our algorithm, contains a coefficient C that, for real-life structures, typically depends both on the flow state and the discharge (cf. Bos 1990). In our algorithm we allow for different values of C for different flow states, but we do not let C depend on the discharge in each separate flow state. This choice was made because it results in equations that can be solved without the iteration steps. Equations with a Q dependent C would probably need some form of iteration in the solution process.

Our choice implies that we will need measurements of the structure in each flow state to determine the actual values for these constants. In the section on our experiments we will discuss a procedure to determine C for the different flow states. For now, we assume we have available the following values for C: $C_{\text{weir,free}}$ for free weir flow, $C_{\text{weir,drowned}}$ for drowned weir flow, $C_{\text{gate,free}}$ for free gate flow, and $C_{\text{gate,drowned}}$ for drowned gate flow.

The computer program using our algorithm needs to calculate a discharge for given upstream (h_1) and downstream (h_3) water levels. Note that we assume that these water levels correspond to subcritical flow. We start by asking a series of questions to determine which flow state best matches the upstream and downstream water level. However, the aim of the algorithm is to supply a discharge, not to signal the precise moment of change of state.

Question: Can the structure be in free weir flow (A1)?
Answer: Only if two conditions are met, the gate must not touch the water surface and the balance of forces between Cross Sections 2 and 3 must be possible. The first condition is easily checked. By combining Eq. (2*a*) with $C = C_{\text{weir,free}}$ and Eq. (3) we obtain a cubic polynomial [Eq. (7)]

$$h_c^3-\left(\frac{B_1 d_1}{B_2}\right)^2(1+2C^2)h_c+2C^2\left(\frac{B_1 d_1}{B_2}\right)^2 h_1=0 \tag{7}$$

The smallest positive root is the critical depth (h_c). If this value is larger than the gate opening flow state A1 is impossible and we need to choose between states B1, B2, and C.

A check of the second condition is somewhat more difficult, we need to use the momentum balance. To do this we need the force on the weir. The upper bound for this force is the static force corresponding with the water level h_3. This force is given by Eq. (3*a*). We can combine this equation with Eq. (2*a*) with $C = C_{\text{weir,drowned}}$ into a fourth-order polynomial [Eq. (8)]

$$\left(\frac{B_2}{B_1 d_1}\right)^2 h_2^4-4C^2\frac{B_2}{B_3 d_3}h_2^3+\left(4C^2\frac{B_2 h_1}{B_3 d_3}+4C^2-\left(\frac{B_2 h_3}{B_1 d_1}\right)^2-1\right)h_2^2 -4C^2 h_1 h_2+h_3^2=0 \tag{8}$$

If Eq. (8) has a root between 0 and h_c then the momentum balance allows critical flow. Now, if $h_c<w$ then we have free weir flow A1 else we need to check for flow states B1, B2, or C.

If on the other hand the first positive root of Eq. (8) lies between h_c and h_3 then either this root is smaller than the gate opening *w* and we have drowned weir flow (A2) or the root exceeds *w* and the flow state is B1, B2, or C.

At this point we either know the flow state (A1 or A2) or we need to ask an additional question to choose between B1, B2, and C.

If the flow state is still unknown then we need to pose the next question: Can this be a case of drowned gate flow. In other words, do we need a water level at Cross Section 2 (h_2) higher than the bottom of the gate to obtain momentum balance?

To answer this question we use Eq. (6*a*) with ζ as in Eq. (6*b*) to calculate the force on the weir. The combination of Eqs. (6*a*) and (6*b*) with Eqs. (2*a*) and (2*b*) results in a quadratic polynomial in h_2 [Eq. (9)]

$$h_3^2-h_2^2+2a\zeta(h_3-h_2)=4C^2 w\left(1-\frac{B_2 w}{B_3 d_3}\right)\frac{h_1-h_2}{\left[1-\left(\frac{B_2 w}{B_1 d_1}\right)^2\right]} \tag{9}$$

Assuming drowned flow, i.e., $C=C_{\text{gate,drowned}}$, we check whether Eq. (9) has a real root greater than *w*. If such a root exists then we are in State B2 or C, if it does not we are in State B1.

We now know the flow state. For a given flow state we calculate the discharge as follows:

- Flow state A1: use Eq. (2*a*) with $C=C_{\text{weir,free}}$ and h_2 given by Eq. (3*b*),
- Flow state A2: use Eq. (2*a*) with $C=C_{\text{weir,drowned}}$ and h_2 equal to the root of Eq. (8) that lies between h_c, and h_3,
- Flow state B1: use Eq. (2*a*) with $C=C_{\text{gate,free}}$ and $h_2=w$, and
- Flow state B2 or C: use Eq. (2*a*) with $C=C_{\text{gate,drowned}}$ and h_2 given by the root of Eq. (9) that lies between *w* and h_3. The procedure is summarized in the algorithm shown in Fig. 5.

Laboratory Setup

Laboratory tests have been performed to verify the formulas and algorithm as described above. Fig. 6 shows the dimensions of the Crump-de Gruyter model used for these tests. Note that the gate is movable. The width of the laboratory canal did not leave room for the horizontal contraction indicated by de Gruijter (1926, 1927a) in his original design. Our laboratory flume was 0.40 m wide and approximately 14 m long. It had a depth of 0.40 m. The model structure was placed roughly halfway in the flume. A sharp-crested movable weir at the downstream end of the canal controlled the downstream water level. Discharge measurements were performed with an ISO standard Rehbock weir. Measurements were done for more than 300 different combinations of discharge, downstream depth and gate position.

$r_c \equiv$ *root of Eq.(7) with* $C \equiv C_{weir,free}$ *between 0 and* h_1
If *Eq. (8) has a root* r_{wd} *with* $r_c < r_{wd} < h_3$ **then**
$h_2 \equiv r_{wd}, C \equiv C_{weir,drowned}$
Else
$h_2 \equiv r_c, C \equiv C_{weir,free}$
End if
If $h_2 < w$ **then**
Use Eq. (2a) to calculate Q
Else if *Eq.(9) has a root s such that* $w < s < h_3$ **then**
$h_2 \equiv s, C \equiv C_{gate,drowned}$
Use Eq. (2a) to calculate Q
Else
$h_2 \equiv w, C \equiv C_{gate,free}$
Use Eq. (2a) to calculate Q
End if

Fig. 5. Algorithm

Comparison of Theory and Experiment

First we give error estimates for the measurements. Then we discuss the choice of the constant *C* in Eq. (2) and the effect of

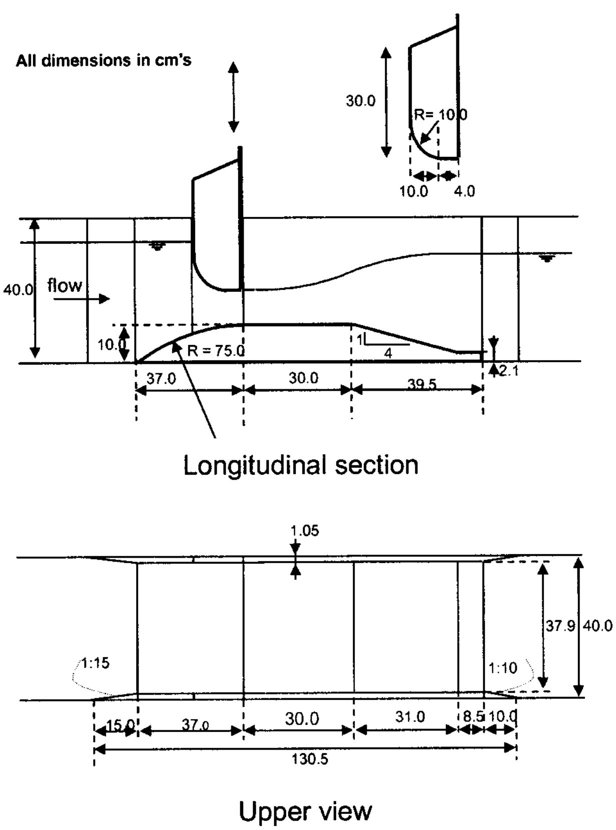

Fig. 6. Model structure used in our experiments

different choices for ζ in Eq. (6). Next, we compare our experimental results with the predictions of the algorithm for the chosen C and ζ.

Error Estimates

Let us first give error margins on the measurements. Most water levels were determined through the use of point gauges with scales that give readings up to 0.1 mm. The weir in the model structure was not entirely flat, there was a slight variation in height between front and back (at most 1 mm). The error in water level measurement was experimentally determined to be approximately 0.5 mm. The bottom and weir level could be established with higher accuracy, 0.3 mm. Assuming normal distribution of the errors and combining the separate errors yields an error of 0.6 mm for the individual measurements and 0.8 mm for the difference between upstream and downstream levels.

The downstream water level was in almost all cases determined by pitot tube. This was due to turbulence in the flow, only a few measurements for high downstream water levels in the case of the drowned weir could be done by point gauge. Repeated readings of the water level and reference level for the pitot tube resulted in an estimate of the observational error of 0.5 mm for the water level and 0.7 mm error for the reference level. Again assuming a normal distribution, the resulting errors become 0.9 mm in the level in the flume and 1 mm in the difference in water levels in the flume.

The discharge measurement by Rehbock weir has a relative error of 1.5%, 0.7% due to a 1% error in the discharge coefficient [ISO 1438-1975(E)], 0.2% due to a $+/-1$ mm error in the width and 0.7% due to errors in measuring the height of the water over the crest. All the above estimates are consistent with the observed spread in the measurements. The spread in discharge measurements runs from 0.5 to 1%, which is in accordance with this estimate, although in some cases the error may also have been caused by a slight fluctuation in the discharge itself. The error in the measurement of the gate opening is estimated to be 1 mm.

Choice of C in Eq. (2)

As mentioned earlier, we use four different values for C one for each of the flow conditions A1, A2, B1, and one for B2 and C together. We determined these as follows. We took the measurements for the middle of the range discharge of 35 L/s. For free flow, while there is a dependence on discharge, this is relatively small (plus or minus 5%), so this choice seems justified. For drowned weir flow, Fig. 3 shows we will have a problem establishing an accurate C whatever method we choose, so a constant C is as good or as bad as any other approximation derived from the measurements. For drowned gate flow the situation is similar to that for free flow.

We calculated $C_{\text{weir,free}}=0.80$ from the measurements for free weir flow by substituting the measured discharge for Q, the measured upstream water level for h_1, and the critical depth calculated with Eq. (3b) for h_2 in Eq. (2a). Analogously, we determined $C_{\text{gate,free}}=0.882$ from the measurements for free gate flow by substituting the measured discharge for Q, the measured upstream water level for h_1, and the set gate opening for h_2 in Eq. (2a).

We used the measured discharge and upstream and downstream water level in Eq. (3a) to calculate h_2 and then we used Eq. (2a) to find $C_{\text{weir,drowned}}=0.93$. In the case of the drowned gate we inserted the measured discharge and upstream and downstream water levels in Eqs. (6a) and (6b) and solved for h_2. The result, when combined with the measurements and inserted in Eq. (2a) resulted in $C_{\text{gate,drowned}}=0.85$.

Boundary Between Free and Drowned Flow

In the previous section we assumed that there is a clear boundary between free and drowned flow. In practice, this boundary is more difficult to identify, the measurements of the water level at Cross Section 2 near the transition from free flow to drowned flow are either inaccurate or impossible due to turbulence. As a result, the simple mathematical criteria used in the algorithm, where we compare calculated h_2 values with either the critical depth or the gate opening cannot be applied to the experiments. An alternative method—better suited to our description of the drowned flow states—is to identify the transition by looking at the point where, for increasing downstream water level, the upstream water level starts to increase (i.e., start of upstream propagation of downstream disturbances). Evidently, this does not give a sharp boundary (a one millimeter fluctuation does not necessarily signal the transition region), but it allows a rough division into definitely free and definitely not-free flow (cf. Table 1, Experiment a, Table 2, Experiment b). This is the separation we used in our calculations for C in the previous section.

In Fig. 3 we see a plot of experimental values of head loss versus downstream water depth for the transition from free to drowned weir flow. It shows a transition (over a range of 10 to 15 cm change in downstream water level) from distinct free flow states for different discharges to a kind of collective drowned flow state with a common asymptotic value for the head loss. This prevents discharge calculations unless we have additional measurements of the flow state over the weir. In computer simulations this information would not normally be available. In Fig. 7, we see a plot of—among other things—the experimental values of head loss versus downstream water depth for the transition from free to drowned gate flow. It shows that, for gates, the head loss increases after reaching a minimum near the transition point from free to drowned flow. The bottle neck under the gate somehow induces additional energy loss and allows discharge calculations for drowned gate flow.

Need for Eq. (6a)

The need for Eq. (6a) with ζ as given by Eq. (6b) can best be shown by considering the two obvious alternatives, $\zeta=0$ and $\zeta=1$.

If we change ζ, then the formula for h_2 changes, so new values of C for free and drowned flow need to be determined for each case. The need for ζ (and the absence of problems due to our calibration of C with 35 L/s data) is seen in a plot of the predicted head loss as a function of downstream depth for 19 L/s for the three different cases (Fig. 7). The predicted head loss is calculated by using Q as known input instead of h_1 and then more or less running the algorithm in reverse to find h_1. When compared with the actual head loss measurements the minimum head loss for $\zeta=1$ lies too high and to the left of the true minimum, while the minimum for $\zeta=0$ lies too low and to the right. The plots for 27 and 35 L/s (not shown) give similar results; for 43 and 51 L/s (not shown) the free flow head loss is overestimated (for all ζ). At 51 and 53 L/s the location of the minimum is no longer clear enough to choose one ζ over another.

Based on observed effects of upstream water level change and the plots of head loss versus downstream water level for the other discharges (not included) it appears that the minimum head loss corresponds to the transition between free and drowned flow.

Table 1. Predicted and Measured Flow Conditions: Experiments a to c

Quantity unit	Measured discharge Q (m^3/s)	Upstream depth d_1 (m)	Downstream depth d_3 (m)	Gate opening w (m)	Upstream level h_1 (m)	Critical depth hc (weir) (m)	Downstream level h_3 (m)	Predicted discharge (m^3/s)	Calculated error in discharge (%)	h_1-h_3 (m)	Relative error in h_1-h_3
Error (+/−)	1.5%	0.0008	0.0009	0.001	0.001	1.0%	0.001	—	—	0.001	—
Experiment (1) transition	0.0271	0.223	0.155	0.400	0.122	0.081	0.054	0.0274	−0.8	0.069	1.5%
from free weir flow	0.0271	0.223	0.170	0.400	0.122	0.080	0.069	0.0274	−1.1	0.054	1.9%
to drowned weir flow	0.0271	0.224	0.194	0.400	0.123	0.080	0.093	0.0274	−1.2	0.030	3.3%
	0.0271	0.224	0.213	0.400	0.123	0.080	0.112	0.0276	−1.8	0.011	9.2%
	0.0271	0.224	0.214	0.400	0.123	0.080	0.113	0.0277	−2.3	0.010	9.7%
	0.0271	0.225	0.217	0.400	0.124	0.080	0.116	0.0241	10.8	0.009	11.8%
	0.0271	0.226	0.220	0.400	0.125	0.080	0.119	0.0244	10.1	0.007	14.9%
	0.0270	0.229	0.224	0.400	0.128	0.080	0.123	0.0243	9.9	0.005	19.2%
	0.0269	0.260	0.257	0.400	0.159	0.080	0.156	0.0275	−2.2	0.003	33.3%
	0.0269	0.291	0.289	0.400	0.190	0.080	0.188	0.0287	−6.9	0.002	50.0%
	0.0267	0.317	0.316	0.400	0.216	0.080	0.215	0.0309	−15.7	0.002	58.8%
Experiment (3) transition	0.0271	0.223	0.101	0.098	0.122	0.081	0.000	0.0274	−0.8	0.122	0.8%
from free weir flow	0.0272	0.223	0.101	0.087	0.122	0.081	0.000	0.0274	−0.8	0.122	0.8%
to free gate flow	0.0271	0.226	0.101	0.076	0.125	0.080	0.000	0.0263	2.8	0.125	0.8%
	0.0271	0.236	0.101	0.065	0.135	0.080	0.000	0.0263	2.7	0.135	0.7%
	0.0270	0.254	0.101	0.057	0.153	0.080	0.000	0.0268	0.7	0.153	0.7%
	0.0269	0.268	0.101	0.053	0.167	0.080	0.000	0.0269	0.0	0.167	0.6%
	0.0269	0.293	0.101	0.048	0.192	0.080	0.000	0.0273	−1.7	0.192	0.5%
	0.0268	0.317	0.101	0.044	0.216	0.080	0.000	0.0272	−1.7	0.216	0.5%
	0.0267	0.339	0.101	0.041	0.238	0.080	0.000	0.0271	−1.7	0.238	0.4%
	0.0266	0.359	0.101	0.039	0.258	0.079	0.000	0.0272	−2.1	0.258	0.4%
Experiment (2) transition	0.0271	0.263	0.152	0.055	0.162	0.081	0.051	0.0272	−0.3	0.111	0.9%
from free gate flow	0.0271	0.264	0.179	0.055	0.163	0.081	0.078	0.0272	−0.4	0.084	1.2%
to drowned gate flow	0.0272	0.264	0.201	0.055	0.163	0.081	0.100	0.0273	−0.3	0.063	1.6%
	0.0272	0.264	0.222	0.055	0.163	0.081	0.121	0.0273	−0.4	0.042	2.4%
	0.0272	0.264	0.226	0.055	0.163	0.081	0.125	0.0246	9.6	0.039	2.6%
	0.0270	0.271	0.230	0.055	0.170	0.080	0.129	0.0245	9.3	0.041	2.4%
	0.0270	0.278	0.233	0.055	0.177	0.080	0.132	0.0256	5.3	0.045	2.2%
	0.0270	0.282	0.236	0.055	0.181	0.080	0.135	0.0253	6.4	0.046	2.2%
	0.0269	0.308	0.253	0.055	0.207	0.080	0.152	0.0258	4.1	0.056	1.8%
	0.0269	0.334	0.269	0.055	0.233	0.080	0.168	0.0267	0.5	0.065	1.5%
	0.0267	0.355	0.287	0.055	0.254	0.080	0.186	0.0264	1.4	0.069	1.5%

Laboratory Results Compared with Model Predictions

The discharge predicted by our algorithm was compared with 351 measurements. Five different discharges (19, 27, 35, 43, and 51 L/s) were used. The following series of steady flow measurements were performed, in Cases 1, 2, 4, and 5 the downstream water level is the parameter that is varied, for cases 3 and 6 the gate position was adjusted.

1. A series where the first measurement corresponded to free weir flow and the last to drowned weir flow;
2. A series where the first measurement corresponded to free gate flow and the last to drowned gate flow;
3. A series where the first measurement corresponded to free weir flow and the last to free gate flow;
4. A series where the first measurement corresponded to drowned weir flow and the last to drowned gate flow;
5. A series where the first measurement corresponded to free weir flow and the last to drowned gate flow; and
6. A series where the first measurement corresponded to drowned weir flow and the last to free gate flow.

As discussed earlier, the values for the reduction coefficient C were obtained from the 35 L/s experiment, $C_{\text{free weir}}=0.93$, $C_{\text{drowned weir}}=0.80$, $C_{\text{free gate}}=0.882$, $C_{\text{drowned gate}}=0.85$. Measurements where the relative measurement error in the head loss exceeded 20% were excluded. The reason for this is the following: If the measurement accuracy did not allow for accurate measurement of the head loss during the experiment, then comparison of results between experiment and a formula indirectly based on that head loss is unlikely to be of use.

We classified the error in the predicted discharge as follows. If the error was less than 5% or less than two times the relative error in the head loss $[=0.001/(d_1-d_3)]$, where we assume an absolute error of 1 mm, then the error was considered acceptable, else it was considered "large." If the error was greater than 20% and more than two times the relative error in the head loss it was considered "very large." We found 12.8% "large" errors and 0.7% "very large" errors in 282 measurements. "Large" errors occur for the free to drowned gate transition and for the submerged weir to free orifice transition.

The excluded measurements—with the exception of the peak for free weir flow to drowned gate flow—correspond to drowned weir flows. Tables 1 and 2 contain a comparison of predicted and actual flows for 27 L/s.

Table 2. Predicted and Measured Flow Conditions: Experiments d to f

Quantity unit	Measured discharge Q (m^3/s)	Upstream depth d_1 (m)	Downstream depth d_3 (m)	Gate opening w (m)	Upstream level h_1 (m)	Critical depth h_c (weir) (m)	Downstream level h_3 (m)	Predicted discharge (m^3/s)	Calculated error in discharge (%)	h_1-h_3 (m)	Relative error in h_1-h_3 (%)
Error (+/−)	1.5%	0.0008	0.0009	0.001	0.001	1.0%	0.001	—	—	0.001	—
Experiment (5) transition	0.0272	0.224	0.148	0.099	0.123	0.081	0.047	0.0274	−0.8	0.076	1.3
from free weir flow	0.0272	0.224	0.183	0.099	0.123	0.081	0.082	0.0274	−0.9	0.041	2.5
to drowned gate flow	0.0273	0.227	0.221	0.099	0.126	0.081	0.120	0.0245	10.3	0.006	15.6
	0.0271	0.258	0.255	0.099	0.157	0.080	0.154	0.0127	53.2	0.003	40.0
	0.0269	0.299	0.287	0.099	0.198	0.080	0.186	0.0256	4.8	0.013	7.8
	0.0269	0.310	0.295	0.099	0.209	0.080	0.194	0.0266	0.9	0.015	6.8
	0.0268	0.318	0.303	0.099	0.217	0.080	0.202	0.0267	0.2	0.015	6.5
	0.0268	0.327	0.311	0.099	0.226	0.080	0.210	0.0274	−2.4	0.017	6.0
	0.0268	0.335	0.318	0.099	0.234	0.080	0.217	0.0278	−3.8	0.018	5.6
	0.0267	0.345	0.327	0.099	0.244	0.080	0.226	0.0274	−2.5	0.018	5.6
	0.0268	0.352	0.334	0.099	0.251	0.080	0.233	0.0270	−0.9	0.018	5.6
Experiment (4) transition	0.0271	0.254	0.250	0.143	0.153	0.080	0.149	0.0282	−3.9	0.004	27.0
from drowned weir flow	0.0270	0.270	0.269	0.143	0.169	0.080	0.168	0.0173	35.9	0.001	83.3
to drowned gate flow	0.0269	0.287	0.285	0.143	0.186	0.080	0.184	0.0203	24.4	0.002	47.6
	0.0269	0.303	0.300	0.143	0.202	0.080	0.199	0.0230	14.3	0.003	31.2
	0.0268	0.318	0.314	0.143	0.217	0.080	0.213	0.0266	0.7	0.005	20.8
	0.0267	0.328	0.324	0.143	0.227	0.080	0.223	0.0241	9.6	0.004	23.3
	0.0267	0.338	0.333	0.143	0.237	0.080	0.232	0.0265	0.5	0.005	18.2
	0.0268	0.350	0.343	0.143	0.249	0.080	0.242	0.0290	−8.5	0.007	14.3
	0.0266	0.361	0.354	0.143	0.260	0.079	0.253	0.0282	−5.9	0.007	14.3
	0.0266	0.375	0.367	0.143	0.274	0.079	0.266	0.0302	−13.4	0.008	11.8
	0.0266	0.385	0.376	0.143	0.284	0.079	0.275	0.0301	−13.0	0.009	11.4
Experiment (6) transition	0.0273	0.255	0.252	0.144	0.154	0.081	0.151	0.0250	8.6	0.003	40.0
from drowned weir flow	0.0273	0.256	0.253	0.116	0.155	0.081	0.152	0.0190	30.3	0.003	34.5
to free gate flow	0.0272	0.261	0.253	0.094	0.160	0.081	0.152	0.0214	21.4	0.008	12.3
	0.0271	0.282	0.253	0.067	0.181	0.080	0.152	0.0244	9.9	0.030	3.4
	0.0268	0.381	0.251	0.040	0.280	0.080	0.150	0.0292	−8.9	0.131	0.8

Note the peak in the error for free weir flow to drowned gate flow. This is the result of the same kind of problem we have with drowned weir flow, the head loss becomes very small. This is probably due to the local minimum in head loss found for the free to drowned gate transition (Fig. 7). This in turn is probably related to the reduction of head loss for increasing downstream water level seen for weir flow (Fig. 3).

Conclusions

For flow conditions where sufficient head loss over the structure is present it is possible to model the discharge of the Crump-de Gruyter gate in terms of upstream and downstream water levels. We found that free weir flow, free gate flow, and drowned gate flow are dealt with correctly. The proposed model consists of a closed set of formulas and is suitable for use in computer simulations. Further research into the approximation of the force on the back of the weir is likely to improve the predictions of the transition points.

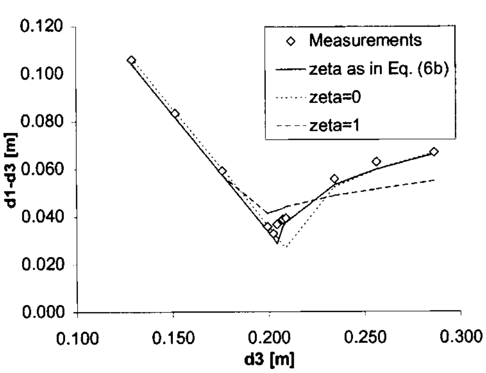

Fig. 7. Head loss versus downstream water level for gate flow

From both experiments and theory it is clear that for a fully drowned weir the head loss over the structure is too small to be used as a means of discriminating between different discharges. In effect, the structure no longer functions as a discharge regulator or measurement device (cf. Bos 1985, p. 66). Even for a Parshall flume the crucial depth over the weir, normally used in combination with upstream and downstream depth to determine the discharge is simply not available if we see the structure as a unit connecting two sections of one-dimensional channel flow simulation. We recommend switching to a formula simulating a head loss based on the contraction and expansion at the structure instead of a gate or weir as soon as fully drowned weir flow occurs. The choice of switching criterion would depend on the purpose of the model and the relative importance of accurate simulation in the case of fully drowned weir flow.

Notation

The following symbols are used in this paper:

a = height of crest of weir relative to channel bottom (0.10 m);

B = width of cross section (0.40 m for channel, 0.379 m at weir);
C = coefficient expressing reduction of discharge due to energy losses (dimensionless number);
d = water depth;
$d_1 = h_1 + a$ = water depth at cross section one (Fig. 2);
$d_2 = h_2$ = water depth at cross section two;
$d_3 = h_3 + a$ = water depth at cross section three (Fig. 2);
F = force;
g = gravitational acceleration (9.81 m/s^2);
H = energy head relative to crest of the weir;
h = water level relative to crest of weir;
h_c = water level relative to crest of weir corresponding to critical depth [Eq. (3b)] over weir;
L = length of crest of weir;
Q = discharge;
w = gate opening;
ΔE_{ij} = energy loss between sections i and j;
ζ = factor representing variation in water level over tail of weir; and
ρ = fluid density.

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