

Free-surface flow simulations for discharge-based operation of hydraulic structure gates

C. D. Erdbrink, V. V. Krzhizhanovskaya and P. M. A. Sloot

ABSTRACT

We combine non-hydrostatic flow simulations of the free surface with a discharge model based on elementary gate flow equations for decision support in the operation of hydraulic structure gates. A water level-based gate control used in most of today's general practice does not take into account the fact that gate operation scenarios producing similar total discharged volumes and similar water levels may have different local flow characteristics. Accurate and timely prediction of local flow conditions around hydraulic gates is important for several aspects of structure management: ecology, scour, flow-induced gate vibrations and waterway navigation. The modelling approach is described and tested for a multi-gate sluice structure regulating discharge from a river to the sea. The number of opened gates is varied and the discharge is stabilized with automated control by varying gate openings. The free-surface model was validated for discharge showing a correlation coefficient of 0.994 compared to experimental data. Additionally, we show the analysis of computational fluid dynamics (CFD) results for evaluating bed stability and gate vibrations.

Key words | computational fluid dynamics, discharge sluice, free-surface flow, gate operation, hydraulic gates, hydraulic structures

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NOTATION

A	amplitude of gate vibration (m)	h_0	upstream water depth before reaching pier (m)
A_{lake}	surface area of lake (m ²)	h_1	upstream water depth between piers, upstream of gate (m)
a	gate opening (m)	h_2	water depth in control section (m)
C_c	contraction coefficient for flow past sharp-edged underflow gate (–)	h_3	water depth downstream of gate, between piers, behind recirculation zone (m)
$C_{c,\text{in}}$	contraction coefficient for flow entering upstream section between piers (–)	h_4	downstream water depth beyond pier (m)
C_D	discharge coefficient for submerged flow (–)	h_{target}	target lake level to be reached at the end of discharge period (m)
C_D^*	average value of C_D over one discharge event, computed by discharge model (–)	k	turbulent kinetic energy (m ² /s ²)
C_E	discharge coefficient as used by Nago (1983) for experimental data	K_P, K_I, K_D	gain parameters of PID discharge controller (–)
e	error value of PID discharge controller (m ³ /s)	m	number of gates opened fully or partially (–)
f_{gate}	frequency of gate vibration (Hz)	n	total number of gates of the structure (–)
Fr	Froude number (–)	\vec{n}	normal vector (–)
g	gravitational constant (m/s ²)	p	pressure (Pa)
h	water depth (m)	Q	discharge (m ³ /s)

Q_{DM}	discharge through one gate, calculated by discharge model (m^3/s)
Q_{gate}	discharge through one gate, calculated by the system model (m^3/s)
Q_{MF}	modular flow discharge based on gate under-flow contraction criterion (m^3/s)
q	discharge per unit width (m^2/s)
t	time variable (s)
U	magnitude of flow velocity vector; in 2DV model defined as $U = \sqrt{u_1^2 + u_3^2}$ (m/s)
U_{vc}	magnitude of flow velocity vector in vena contracta (m/s)
$\vec{u} = (u_1, u_3)$	flow velocity vector in 2DV model; u_1 is horizontal velocity, u_3 is vertical velocity (m/s)
Vr	reduced velocity parameter of flow-induced vibrations (-)
V_{tot}	total volume passing the structure in a given amount of time (m^3)
$V_{tot,req}$	required total volume to pass the structure in a given amount of time in order to reach h_{target} (m^3)
w	width between piers (m)
α	calibration parameter for turbulent flow in bed stability parameter (-)
β	relaxation factor in formula for C_D in system model (-)
ε	turbulent dissipation (m^2/s^3)
ξ_{in}	entrance loss coefficient (-)
ξ_{out}	exit loss coefficient (-)
Ψ	stability parameter for beginning of motion of granular bed material (-)
∇u_i	gradient operator, defined as $\nabla u_i = \begin{pmatrix} \partial u_i / \partial x \\ \partial u_i / \partial z \end{pmatrix}$
$\nabla \cdot \vec{u}$	divergence operator, defined as $\nabla \cdot \vec{u} = \partial u_1 / \partial x + \partial u_3 / \partial z$
$\binom{q}{r}$	number of combinations of r objects out of q objects ($0 \leq r \leq q$), defined as $\frac{q!}{r!(q-r)!}$
$[r]$	floor function, defined as $\forall r \in \mathbb{R}, [r] = \max(n \in \mathbb{Z}: n \leq r)$
$\lceil r \rceil$	ceiling function, defined as $\forall r \in \mathbb{R}, \lceil r \rceil = \min(n \in \mathbb{Z}: n \geq r)$
\bar{s}	time-average of quantity s

INTRODUCTION

This paper gives an outline of how near-field free-surface flow simulations can be used in the operation of gates of large hydraulic structures.

Barrier operation is commonly based on water level predictions from system-scale far-field flow models. The procedures are aimed at fulfilling the main function of the structure: for a weir in a river this is to maintain the upstream water level; for a discharge sluice this is to transfer river water out to the sea while keeping a safe inland level. Present-day hydraulic structures have various secondary functions, such as providing favourable ecological conditions, for which usually no numerical aids are available in daily operation. A better prediction of the flow near structures would be beneficial to durable performance of all barrier tasks.

Proper design studies pay attention to all functions of a structure and assess the impact of all relevant flow features. However, operational constraints change in time for natural reasons (e.g. sea-level rise) or political reasons (e.g. ‘Kierbesluit Haringvlietsluizen’) (see [Rijkswaterstaat 2004](#)). In addition, sometimes the design criteria that were originally applied cannot be retrieved, yielding uncertainty about safety levels and allowable limits of gate settings in the present.

There are several aspects in contemporary barrier management for which an informed view on discharge and flow around gates is essential. First, the prediction of bed material stability and scour, including local erosion ([Hoffmans & Pilarczyk 1995](#); [Azamathulla 2012](#)), as well as large-scale morphological changes of surrounding bathymetry ([Nam *et al.* 2011](#)) greatly depend on the flow. Second, ecological issues such as fish migration, salt water intrusion and mobile fauna are also linked with local flow characteristics ([Martin *et al.* 2005](#)). Third, other relevant aspects are the dynamic forces associated with flow-induced gate vibrations ([Naudascher & Rockwell 1994](#)) and the impact of flow around structures on nearby shipping traffic. Fourth, for the structure itself, local flow prediction is useful for dealing with abnormal conditions: downtime of gates during scheduled maintenance or unexpected gate failure.

The barriers and sluices built in the south-west of The Netherlands in the period 1960–2000 are good examples of structures where different functions are combined. Present management of the barriers at Haringvliet and Oosterschelde calls for smart use to allow for regulation of fresh and salt water flows and fish migration. At the same time, the aging process of these structures demands an increasing awareness of structural safety issues. The new storm-surge barrier of Saint Petersburg, Russia, is another example. This large dam houses two sector-gates and three sections of radial gates that protect the low-lying city centre and regulate the discharge from the river Neva. Operation of this complex structure must rely on state-of-the-art flow models.

The above considerations motivate quantification of flow around a hydraulic structure. The aim of this paper is to lay a foundation for numerical models to estimate gate discharges and to evaluate the impact of flow near hydraulic structures in a way that is fit for operational applications. The influence of waves is not investigated; the focus is on flow (current).

Traditionally, flow around hydraulic structures is studied experimentally in the design stage or as a fundamental research topic (Kolkman 1994; Roth & Hager 1999). Numerous numerical studies have looked into sluice gate flow (Khan *et al.* 2005; Kim 2007; Akoz *et al.* 2009), but no single accepted, validated modelling tool exists for assessing turbulent gate flow with suitable practical value. Estimating discharge over weirs or under gates is not trivial. New discharge equations are still being introduced, both from informatics viewpoints (Khorchani & Blanpain 2005) and from the traditional viewpoint of measurements (Habibzadeh *et al.* 2011).

System-scale models of inland water systems simulate the flow in river branches by solving the one-dimensional or quasi-two-dimensional Shallow Water equations (Deltares 2012a, b). The fact that these hydrostatic models do not simulate the flow around hydraulic structures explicitly is not a severe limitation for most applications. The system effect of the operation of various gates on the water levels in adjacent water bodies (river branches) can thus be studied (for instance Becker & Schwanenberg 2012). For stability of granular bed material and salt water transport, however, the flow acceleration in the vertical dimension needs to be simulated. Moreover, the downside of primarily water level-centred validation and calibration in combination with parameterized structure representations

(such as constant discharge coefficients) is that the prediction quality of discharges in system-scale models is often unclear. Warmink *et al.* (2008, 2010) investigated the uncertainty in calibration of water levels in river models resulting from the limited availability of discharge data. It was concluded that the necessary extrapolation of the calibration parameter (bed roughness of main channel) leads to significant uncertainty in simulated design water levels. More intensive measurement of discharges, for which most gated structures are ideal, and a physically more realistic representation of hydraulic structures in models are self-evident improvements that nevertheless require a culture shift.

The application of computational fluid dynamics (CFD) in the assessment of flow impact issues that arise long after the start of operation of a structure is rare. Bollaert *et al.* (2012) employ numerical modelling to assess the influence of gate usage on the formation of plunge pool scour of a hydropower dam. For some issues, like salt water intrusion and sediment transport past a discharge-regulating structure, the solution cannot be found in a modelling tool at one scale. The local flow simulation should in those cases be coupled to a mid- or far-field model that covers a larger area.

A central role nowadays is played by the multi-disciplinary field of hydroinformatics (Solomatine & Ostfeld 2008; Krzhizhanovskaya *et al.* 2011; Melnikova *et al.* 2011; Pyayt *et al.* 2011a; Pengel *et al.* 2013), in which different forms of modelling (physics-based and data-driven) are considered and combined with contemporary computational techniques like machine learning (Pyayt *et al.* 2011b). In the context of the present study, it is noted beforehand that for a complex hydraulic structure, data-driven modelling alone is not an apt option, because a single Q - H -relation does not describe all states (Kolkman 1994), or is highly impractical as it would require extensive permanent monitoring.

This study takes the underlying physics as a starting point: elementary flow equations are combined with two-dimensional model in the vertical (2DV) time-dependent CFD simulations. The method bridges modelling scales with a minimum of data coupling and at the same time introduces the use of numerical aids into practical barrier operation for issues that at present are decided upon by expert judgement by the operator.

The remainder of this paper is organized as follows: first, we describe the overall approach, then the method is

described in three sections about discharge modelling, free-surface flow simulation and analysis of the modelling results. Next, the results of a series of validation runs for the free-surface model are discussed, followed by the results of a test case that gives numerical examples of all modelling steps. We end the paper with recommendations, conclusions and an outlook on future work.

APPROACH

For obtaining a timely prediction of the flow around gates, we propose a multi-step physics-based modelling strategy which uses data input from a system-scale model. The work-flow of the suggested gate operation system is shown in Figure 1.

The first step consists of the extraction of predicted water levels on both sides of the structure from a far-field (system-scale) model that contains the structure. Different possible gate settings (when to open, how many gates to use) are identified in the second step. All options need to be assessed in terms of discharge capacity; this happens in step 3. In the fourth step of Figure 1, for all gate configurations capable of discharging the required volume, the resulting flow is simulated using CFD. Subsequent analysis of the simulation results determines the impact of the flow for specific issues such as bed stability. The fifth and final step comprises the actual decision of gate operation actions. The conventional sequence of steps taken by most operational systems follows the dashed line in Figure 1, skipping steps 3 and 4. The present study focuses on steps 2–4, which can be seen as an addition to computational decision support systems (steps 1 and 5) by Boukhanovsky & Ivanov (2012) and Ivanov *et al.* (2012).

A multi-gated discharge sluice with underflow gates will be used to describe the modelling method. The central

question addressed is how to find the set of gate configurations capable of delivering the required discharge that also meet the relevant constraints on flow properties.

METHOD

Discharge computations

Configurations of multi-gated structure

Let us consider the gate configurations of a discharge structure consisting of n similar openings, each accommodating a movable gate, see Figure 2. In its idle state, all n gates close off the openings between the piers and the total discharge is zero. During a discharge event, m gates will be opened partially or completely, allowing a certain discharge through the structure. A ‘gate configuration’ is defined as the allocation of a number of gates ($m \leq n$) that are opened with a gate opening $a(t)$ while the other gates remain closed. All gates selected for opening will be operated similarly, i.e. with the same $a(t)$.

Before deciding which gates to open, first the possible combinations of opening gates are identified and counted. In general, flow instabilities are not favourable for maintaining an efficient and controllable discharge. As in other parts of physics, symmetry is a global measure for stability of free-surface flows. If asymmetry is allowed, m gates can be chosen freely from the total of n available slots. Then the number of possible combinations is obviously $\binom{n}{m}$, using the common notation for combinatorial choice of m objects out of n . For the condition of symmetry to hold, gates may only be opened in such a way that the pattern is symmetric about the vertical plane of symmetry in flow direction (see Figure 2). This implies that the number of options reduces

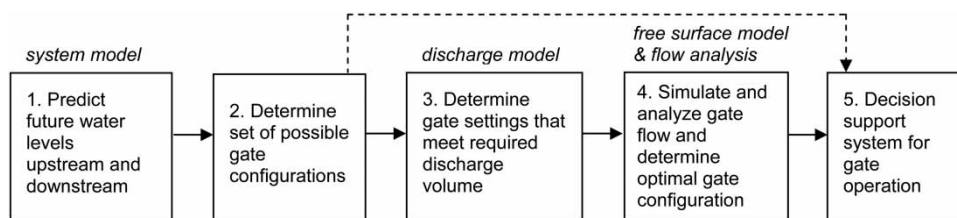


Figure 1 | Scheme of evaluation steps leading to a decision on optimal gate operation. Steps 1–4 are treated in this paper. The dashed line shows the shorter decision sequence taken by barrier systems that do not take into account flow effects.

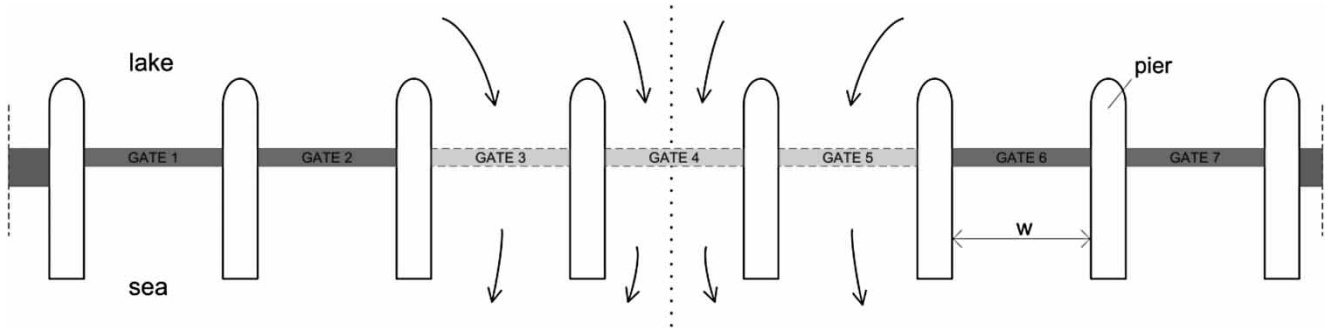


Figure 2 | A multi-gated discharge sluice in plan view. In this example, gates 3, 4 and 5 are opened, the others are closed; so $n = 7$ and $m = 3$. The dotted line depicts plane of symmetry.

to $\binom{\lfloor n/2 \rfloor}{\lfloor m/2 \rfloor}$ for all $0 \leq m \leq n$, where m cannot be chosen odd if n is even – in which case there are no options at all.

For a structure with seven gates ($n = 7$), for instance, the total number of possible ways to open 1, 2, ..., 7 gates is $\sum_{i=1}^7 \binom{7}{i} - 1 = 2^7 - 1 = 127$ if asymmetry is allowed and $\sum_{i=1}^7 \binom{\lfloor 7/2 \rfloor}{\lfloor i/2 \rfloor} = 2^{\lfloor 7/2 \rfloor} - 1 = 15$ if only symmetric configurations are permitted.

This shows that the symmetry constraint greatly reduces the number of ways to open a given number of gates. Furthermore, an even number of gates has roughly half the number of possibilities, because opening any odd number of gates then results in asymmetric inflow. This could also hold for an odd-numbered gate structure which misses one (or any odd $m < n$) of the gates due to maintenance or operational failure.

System model and gate control

The basis is formed by a classic box model, see for example [Stelling & Booij \(1999\)](#). The focus is on submerged flow through a multi-gated outlet barrier that blocks seawater from entering the lake at high tide and discharges river water to sea at low tide, see [Figure 3](#). This basic model serves in the present study as a surrogate system-scale model. The water levels it generates will be used as boundary conditions for the near-field modelling.

Assuming barrier gates are closed (except when discharging under natural head from lake to sea) and assuming zero evaporation and precipitation, the system is described by:

$$Q_{\text{river}} - Q_{\text{barrier}} = A_{\text{lake}} \frac{dh_{\text{lake}}}{dt},$$

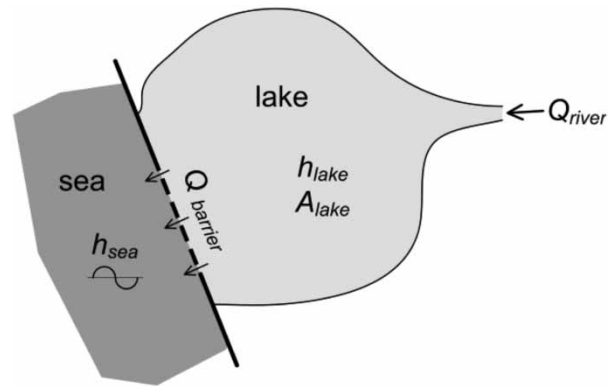


Figure 3 | Classic box model of outflow of a river to sea. An outlet barrier structure regulates the lake level while keeping salt seawater out.

where Q_{river} is discharge from a river, Q_{barrier} is the total discharge through the gates of the barrier, h_{lake} is the water level in the lake, A_{lake} is the area of the lake assumed independent of h_{lake} .

Submerged flow past an underflow gate is by definition affected by the downstream water level. The associated discharge depends on both water levels (sea and lake), the gate opening a and a discharge coefficient for submerged flow C_D . The discharge Q through a barrier gate at time t is written as:

$$Q_{\text{gate}}(t) = C_D a_i(t) w \sqrt{2g(h_{\text{lake}}(t) - h_{\text{sea}}(t))},$$

where w is the flow width (see [Figure 2](#)) and the subscript ‘barrier’ is dropped from now on. Sea level h_{sea} is approximated by a sine function. The total discharged volume that passes the barrier in the period during which $h_{\text{lake}} > h_{\text{sea}}$ is found after summing over all m gates and integrating with respect to time.

Two gate opening scenarios will be considered. In both scenarios equal gate openings $a(t)$ are applied to all m gates selected for opening. The first scenario uses a constant gate opening a_{const} for the whole discharge period (from t_{start} to t_{end}). The opening required to lower the lake level to a desired lake level h_{target} is found by estimating the average required discharge $Q_{tot,req}$ to achieve this and by making estimates of the average discharge coefficient and water levels during the discharge period:

$$a_{const} = \frac{\bar{Q}'_{tot,req}}{m\bar{C}'_D w \sqrt{2g(\bar{h}'_{lake} - \bar{h}'_{sea})}} \text{ with}$$

$$\bar{Q}'_{tot,req} = \frac{A_{lake}(h_{lake}(t_{start}) - h_{target})}{t'_{end} - t_{start}}$$

where bars are time-averages and primes indicate predictions of future values. In the second scenario, the discharge is regulated by a proportional integral derivative (PID) controller (Brown 2007). The goal of this scenario is to have a more constant gate discharge by varying the gate openings in time, whilst still achieving the same h_{target} as in the first scenario. The discrete PID formula for discharge at t_i is:

$$Q(t_i) = K_P e(t_i) + K_I \sum_{j=1}^i e(t_j) + K_D \frac{e(t_i) - e(t_{i-1})}{\Delta t},$$

where K_i are the gain parameters and the error value is defined as $e(t_i) = Q_{set} - Q(t_{i-1})$. In the simulations, $K_P =$

0.10, $K_I = 0.45$ and $K_D = 0.55$ are used. The setpoint Q_{set} is constant and equal to $Q_{tot,req}$, except for linear setpoint ramping applied at the start of discharge to prevent undue fluctuations of gate position. At each time step, the required gate opening is derived from this discharge divided by $mC'_D w \sqrt{2g(h_{lake}(t) - h_{sea}(t))}$. Figure 4 shows the flow chart of the system model. It includes computations of the two gate operation scenarios.

Figure 4 shows that the total discharge computed by the system model Q_{tot} is being used to calculate the new lake level. Additionally, it shows that at the start of each discharge event, i.e. when the gates are opened, the prediction of the discharge coefficient C'_D is updated using data from the discharge model. For both situations, with and without PID control, this coefficient is found by a relaxation function with the mean discharge coefficient C^*_D of the previous discharge event computed by the discharge model. For the n th discharge event, the update formula reads:

$$C'_D(n) = C'_D(n-1) + \beta \{C^*_D(n-1) - C'_D(n-1)\}$$

In all computations, a relaxation factor $\beta = 0.75$ is applied.

Discharge coefficients actually depend on numerous factors. Also, flows through neighbouring gates influence each other. To distinguish between different gate configurations with the same total flow-through area $m \cdot w \cdot a_{const}$, these two things need to be taken into account. This is done in the discharge model described in the next subsection, see also the bold block in Figure 4.

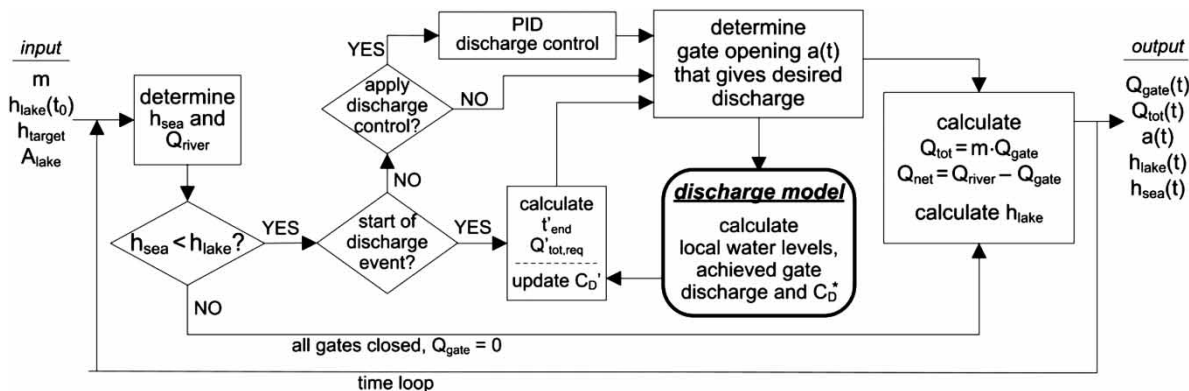


Figure 4 | System model: flow chart of gate control and water level computations.

Discharge model

Vertical lift gates with underflow are raised vertically between piers of a structure. The two main flow types that occur are free flow and submerged flow. When the gates are lifted higher than the water surface, there exists free or submerged Venturi flow (Boiten 1994). All flow types have different discharge characteristics and associated formulae. For estimating the submerged flow discharge, the local water depths are schematized according to Figure 5 (after Kolkman 1994). Conservation of the energy head (Bernoulli equation) is applied in the accelerating parts and the momentum equations in the decelerating parts, yielding a system of four equations (see Appendix, available online at <http://www.iwaponline.com/jh/016/215.pdf>).

Transitions h_0-h_1 and h_3-h_4 with loss coefficients ξ_{in} and ξ_{out} represent the effects of flow entering and leaving the narrow area between two piers. Transitions $h_1-h_2-h_3$ are the characteristic underflow gate zones, see Battjes (2001) for details. Computations were carried out according to the flow chart shown in Figure 6 with the aim of giving better discharge estimates. The lake and sea levels computed in the system model served as boundary conditions – for variables h_0 and h_4 of this model, respectively.

A good geometric design of a discharge-regulator is such that no transition occurs from one flow type to another during regular usage. The model therefore checks if indeed submerged discharge occurs. As criterion for reaching the modular flow discharge Q_{MF} , the minimum flow depth in the control section h_2 is compared to the flow height in the point of maximum vertical contraction $C_c \cdot a$, the so-called ‘vena contracta’. Free and intermediate flow regimes are thus detected, but not calculated. Submerged Venturi flow is not considered either, since the idea is to actively control the flow.

All four non-linear equations are reshaped into third-order polynomials $f(h_i, h_{i+1}, Q) = 0$. Discharge Q is substituted for the velocity terms and remains as the only unknown in the system of equations. As prescribed for sub-critical flow conditions (Chow 1959), computational direction behind the gate is from downstream to upstream (h_4 to h_2). On the lake side, computations go in flow direction up to the control section (h_0 to h_2). The discharge coefficient C_D is derived from the contraction coefficient C_c for sharp-edged gates, fitted on experimental data cited in Kolkman (1994) so that the full range of gate openings a/h_1 is covered; see the Appendix for equations (available online at <http://www.iwaponline.com/jh/016/215.pdf>).

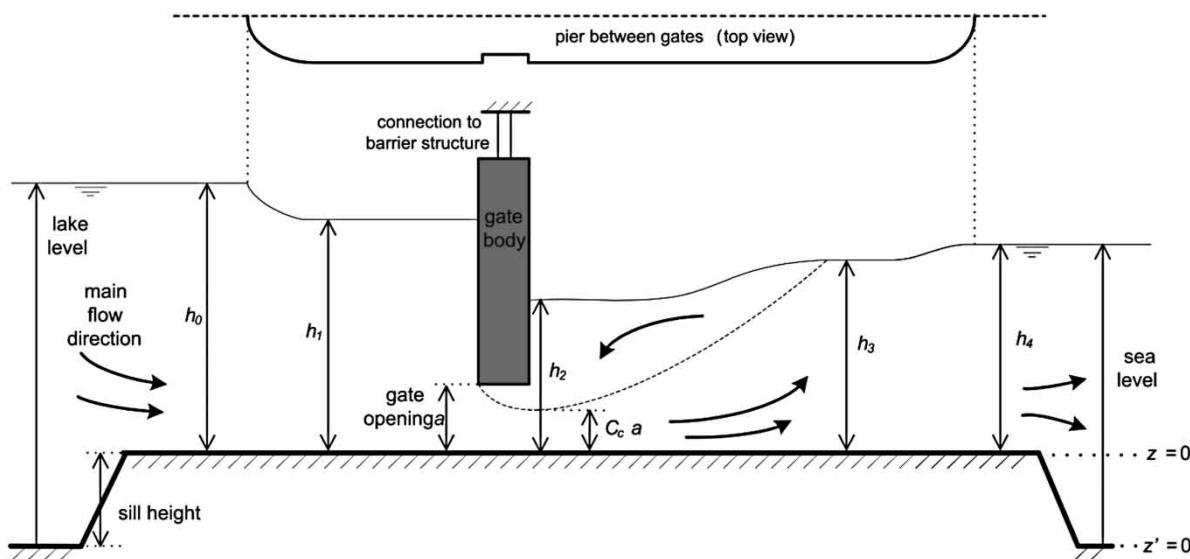


Figure 5 | Definitions of local water depths h_i for underflow gate in hydraulic structure, after Kolkman (1994). Above: top view of pier; below: cross-section free water surface around gate. Sketch not to scale.

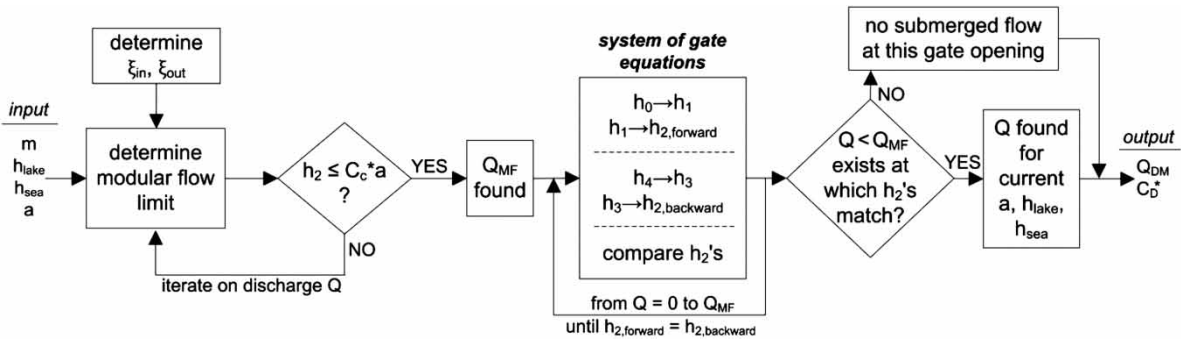


Figure 6 | Flow chart of discharge model. This computation is repeated each time step; it is fully contained in the block named 'discharge model' in Figure 4.

Iterations on Q ultimately yield a value at which $h_{2,forward}$, computed from upstream, is equal to $h_{2,backward}$ computed from downstream. This is the achieved value of Q for the given gate opening a . The entrance and exit losses are assumed to depend on the number of gates in use (m). The method does not distinguish between different gate configurations with equal m , however. Numerical results are shown in the results section.

CFD SIMULATIONS

Step 4 in Figure 1 consists of two parts: free-surface CFD simulations (discussed in this section) and flow analysis (discussed in the next section).

Model set-up

A non-hydrostatic flow model is applied to find out which of the selected gate settings is most favourable in terms of flow properties. The two-dimensional domain (2DV) is defined by a vertical cross-section through the gate section from

one water body (lake) to the other (sea), see Figure 7. A rigid rectangular gate with a sharp-edged bottom is modelled implicitly by cutting its shape out of the flow domain. The Reynolds-Averaged Navier–Stokes (RANS) equations for incompressible flow, included in the Appendix (available online at <http://www.iwaponline.com/jh/016/215.pdf>), are the basis for the simulations. Figure 8 gives the flow chart of the CFD simulations. The model domain covers the flow from h_1 to h_3 . These input values are taken from the discharge model.

For each simulated flow situation, two consecutive runs are made: a steady-state run and a time-dependent transient run. In the former run, iterations on the outflow velocity profile are done until pressure at the surface becomes zero. The results of this pre-run are then implemented as initial conditions for the transient run, which uses a moving mesh to simulate the free surface. Boundary conditions are similar for both runs except for the surface downstream of the gate, see Figure 7.

The upstream flow boundary consists of a hydrostatic pressure profile $p(z) = \rho g(h_1 - z)$. The downstream boundary is a block profile u -velocity. No slip is applied at the

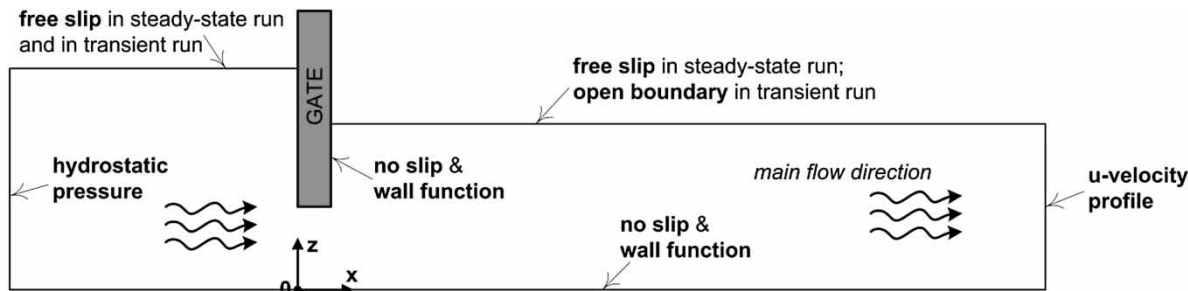


Figure 7 | Boundary conditions of CFD model. The main flow direction is from left to right. Sketch not to scale.

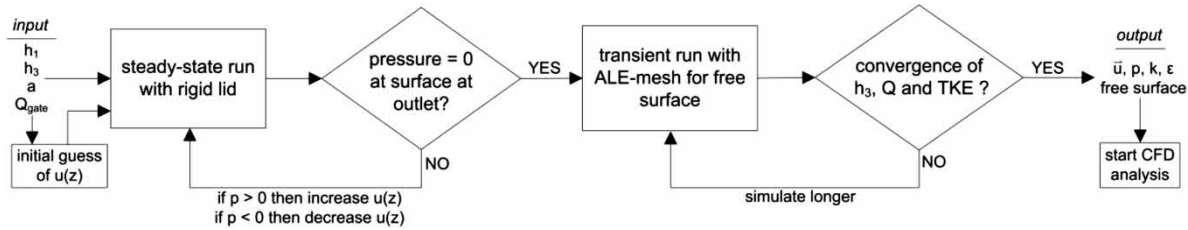


Figure 8 | Flow chart of FEM free-surface flow simulations.

walls ($\vec{u} = 0$) along with a wall function. The steady pre-run uses a ‘rigid lid’ (free slip boundary, $\vec{u} \cdot \vec{n} = 0$) for the downstream water surface. The upstream free surface is modelled as a rigid lid in both runs.

An unstructured computational mesh is used with refinements near the bottom wall and gate boundaries, made up of around 35,000 triangular elements and yielding about 230,000 degrees of freedom for a transient run. Figure 9 shows part of the mesh. The Arbitrary Lagrangian–Eulerian (ALE) method with Winslow smoothing (Donea *et al.* 2004) is applied to compute the deformation of the computational mesh downstream of the gate. At the top boundary in the transient run, the velocity condition is an open boundary with zero stress in normal direction. At the same boundary, the mesh velocity in normal direction is prescribed as $u_{\text{mesh},n} = u_1 \cdot n_x + u_3 \cdot n_z$ (Ferziger & Perić 2002). Mesh convergence tests showed that the applied mesh is sufficiently dense so that results do not improve on further mesh refinement.

The more common choice of applying a velocity condition upstream and a pressure boundary downstream conflicts with the required ALE moving mesh condition at the outlet boundary. Vertical mesh freedom is necessary for the surface movement. A hydrostatic pressure profile cannot be prescribed at the outlet, since any change in

water depth at this boundary would imply a change of local pressure, which contradicts the applied pressure profile.

In the course of the transient run, the free surface adapts to the pressure field and vice versa. Because the physical flow situation is quasi-steady, with fluctuations depending on degree of submergence and gate opening, the surface may show oscillations in time in its equilibrium state. As a consequence, the flow discharge is also not strictly constant in the equilibrium state.

The package Comsol Multiphysics is used to simulate the gate flow. This finite element method (FEM) solver is applied to solve the discretised RANS equations. The generalized alpha time-implicit stepping method is applied to ensure Courant stability, with a strict maximum time step of $\Delta t = 0.02$ s. The time step in the CFD model is completely independent of the time step in the system model and discharge model. The variables are solved in two segregated groups using a combination of the PARDISO solver and the iterative BiCGStab solver in combination with a VANKA preconditioner. The standard k-epsilon model is used for turbulence closure. Simulation of 24 seconds of physical time took around 6 hours of wall-clock time on an Intel 8-core i7 processor, 2.93 GHz, 8 Gb RAM, occupying on average 1 Gb RAM and 50% of total CPU power.

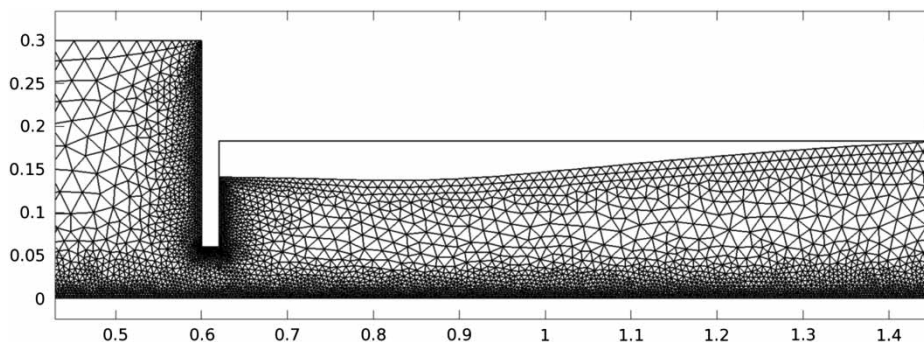


Figure 9 | Example snapshot showing part of the computational mesh. Deformed surface downstream of gate is visible. Flow is from left to right.

ANALYSIS

The second part of step 4 in Figure 1 is the analysis of the modelling results obtained in previous steps. In this section, three aspects of analysis are discussed: flow parameters, vibrations and bed stability.

Flow parameters

Three parameters that are required for assessing various types of flow impact are extracted from the CFD model: the contraction coefficient C_c , the velocity in the vena contracta U_{vc} and the Froude number (Fr). The flow field is interpolated to a regular grid, so that the edge of the separated layer is found, see Figure 10. The contraction coefficient is thus found directly.

The cross-sectional averaged velocity in the vena contracta is defined by a spatial average in the separated shear zone:

$$U_{vc}(t) = \frac{1}{C_c(t)a} \int_{z=0}^{C_c(t)a} U(z, t) dz$$

where U is the velocity magnitude scalar at the point of maximum flow contraction. For gate flow with significant fluctuations, the temporal mean of this quantity, \bar{U}_{vc} , may be used.

The Froude number is a widely used dimensionless measure for flow-related surface curvatures. It is used for describing the transition from intermediate to free flow regimes and predicting modular flow discharge and associated gate openings. Here it is defined as:

$$Fr(t) = \frac{U_{vc}(t)}{\sqrt{gh_2(t)}}$$

in which obviously $h_2 = C_c \cdot a$ in fully free flow. An overview of critical flow theory from a historical perspective is given by Castro-Orgaz & Hager (2010) and from a more practical viewpoint by Boiten (1994). In a more complete flow assessment, not only the vertical contraction caused by the underflow gate is used as a criterion for modular flow, as is done here, but also contraction caused by horizontal and possibly vertical flow domain transitions at the inlet of the structure should be included.

Vibrations

The interaction of current with the movable hydraulic gate is capable of causing significant flow-induced vibrations (FIV). Although dedicated design tests greatly reduce susceptibility for dangerous dynamic forces (Jongeling & Erdbrink 2010), active prediction and control will broaden the windows of operation. The literature on dynamic gate forces caused by this phenomenon uses a dimensionless parameter of reduced velocity to signify occurring gate vibrations (Hardwick 1974; Billeter & Staubli 2000; Erdbrink 2012). In time-dependent form it is written as:

$$Vr(t) = \frac{U_{vc}(t)}{f_{gate}(t) \cdot L}$$

where f_{gate} is the response frequency of the structure in Hz; L is a characteristic length scale of the gate, usually the thickness of the gate bottom, and U_{vc} as defined in the previous section. The response frequency is not easily determined analytically (see general formula in Appendix, available online at <http://www.iwaponline.com/jh/016/215.pdf>); among other reasons because the 'added' water mass m_w that is caused by the inertia of water being pushed away by the gate deviates from analytical values

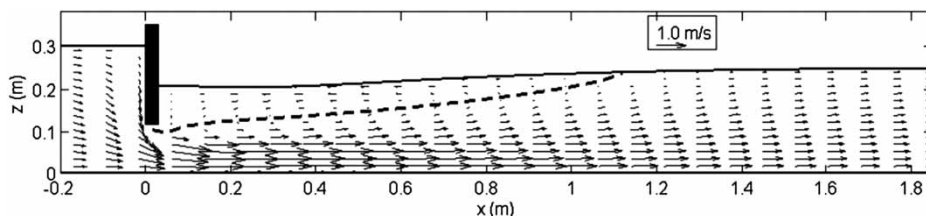


Figure 10 | Vector flow field of run II. Flow is from left to right. The computed free surface behind the gate shows local lowering. Dashed line indicates separation between positive and negative u_1 -velocity. The figure shows only part of the actual computational domain. Total domain length is 3.6 m.

at non-zero gate flow (Blevins 1990). The gate frequency may best be monitored in situ by installing sensors – which need to be sensitive to small amplitudes in order to have predictive value. Erdbrink *et al.* (2012) provide a recipe for a data-driven gate control system for gate vibrations. It is therein proposed to combine physics-based modelling and sensor data with machine learning computations to steer the gates clear of riskful situations.

From numerous experimental studies it is concluded (Naudascher & Rockwell 1994) that for a specific gate, the amplitude A due to FIV, in cross-flow or in-flow direction or both, is a function of Vr , a and submergence:

$$A = f(Vr, a, h_3)$$

Details of the gate geometry are decisive for occurrence or absence of vibrations. A database with response data from past laboratory studies could be used to predict amplitudes of future flow situations in an operational system.

Scour and bed protection

The classical prediction of local scour downstream of weirs and sluice structures caused by outlet currents is described by Breusers (1966) and Hoffmans & Pilarczyk (1995). More recently, contemporary computational techniques were introduced for scour estimation, e.g. Azmathullah *et al.* (2006). In the classical physics-based design formulae, turbulence parameters are used to predict the depth of the scour hole in unprotected beds. For beds protected with granular material (loose rocks), the Shields parameter is a classic non-dimensional measure applied as a first indicator for instability (Shields 1936). An adapted version of this parameter used by Jongeling *et al.* (2003) and elaborated upon by Hofland (2005) and Hoan *et al.* (2011) is defined as:

$$\Psi(x) = \frac{\langle (\bar{U}(x) + \alpha\sqrt{k(x)})^2 \rangle}{\Delta g d(x)} \quad \text{with } \Delta = \frac{\rho_s - \rho_w}{\rho_w},$$

where $\langle \dots \rangle$ denotes spatial averaging over the whole water depth, k is the turbulent kinetic energy (TKE), d is the local water depth, \bar{U} is the mean flow velocity magnitude and α is an empirical parameter for bringing into account

the turbulence (that depends on flow type and local geometry, e.g. slopes in bottom profile).

MODEL VALIDATION RESULTS

A series of validation runs was performed for the free-surface model. ‘Validation run’ is used here in the meaning discussed by Stelling & Booij (1999): the uncalibrated model is run without any tweaking of parameters to see if it can reproduce the most important physical features. Experimental laboratory data by Nago (1978, 1983) for a vertical sharp-edged gate under submerged efflux serve as comparison. Nago’s (1978, 1983) dimensions were used without any scaling. His discharge formula $Q = C_E a w \sqrt{2gh_1}$ does not contain the downstream level h_3 explicitly. Its influence is instead found in the discharge coefficient C_E . The simulated discharge is computed by spatial integration of horizontal velocity at the outflow boundary. In Figure 11, coefficient C_E is plotted for different series of dimensionless gate openings and for a range of dimensionless downstream levels.

The results of the validation runs make clear that the simulations capture the discharges of the experimental data quite accurately: the correlation coefficient is 0.994 and the root mean square error is 1.14%. The fact that the uncalibrated model shows good discharge estimates gives confidence in the predictive power of this modelling approach. Physical output not validated here (such as TKE) may be calibrated in future studies by adjusting suitable model parameters. Convergence of various flow variables occurs at different rates. First, the mean velocities stabilize, and then the forces on the gate converge, then the discharge, and lastly the turbulent energy.

The chosen boundary conditions proved to lead to stable results for all submergence ratios of Nago’s (1978, 1983) data. It was found that the moving mesh is the critical factor for numerical stability. ALE is a suitable method for computing the free surface for quasi-steady gate flow as long as the flow remains submerged. Steep surface gradients associated with lowering h_3 cause inverted mesh elements and hence numerical instabilities.

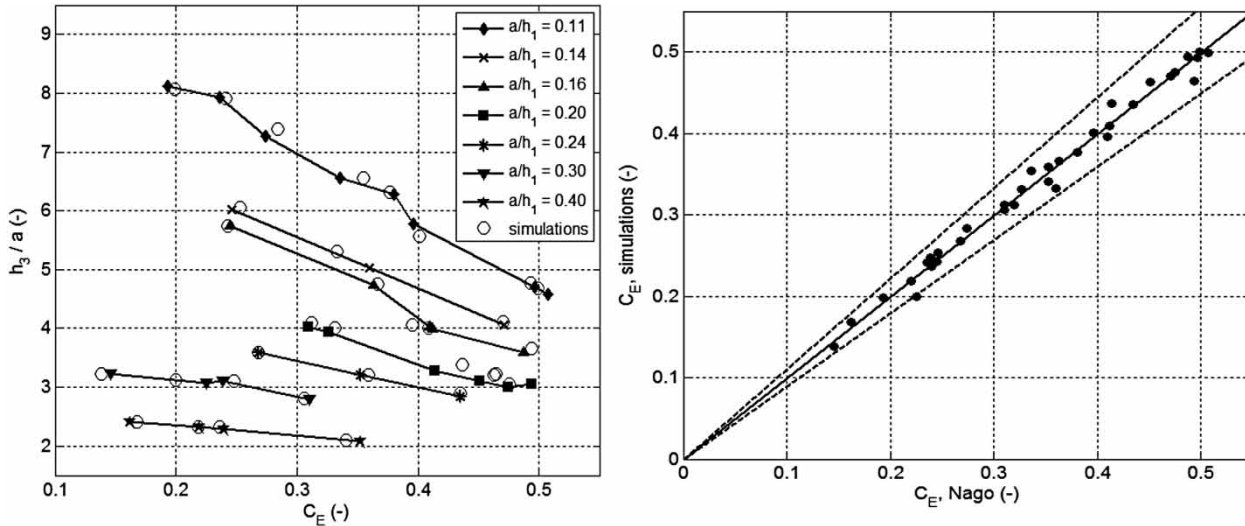


Figure 11 | Results of validation runs showing discharge coefficient C_E simulated by the free-surface CFD model versus experimental data of submerged flow of a sharp-edged underflow gate by Nago (1978, 1983). Left: sorted by gate opening (a/h_1) and downstream level (h_3/a). Right: direct comparison of the same data. Dashed lines mark 10% deviation.

TEST CASE RESULTS

The described methods are illustrated by a test case example. The results of three modelling steps are discussed: the sluice model containing the system model (for water levels) plus the discharge model (Figures 4 and 6), the free-surface model (Figure 8) and analysis of vibrations and bed stability. Four tidal cycles and four discharge events were modelled for a discharge sluice with seven gates regulating a lake with constant river inflow. The goal of the computations is to determine the optimal number of gates to open and the best gate operation scenario.

Results of system and discharge model

Model parameters

- $n = 7, m = 1, \dots, 7$
- $A_{\text{lake}} = 1.9 \cdot 10^7 \text{ m}^2$
- $Q_{\text{river}} = 100 \text{ m}^3/\text{s}$
- $h_{\text{lake}}(t = 0) = 6.1 \text{ m}$
- $h_{\text{target}} = 6.0 \text{ m}$
- $w = 22.5 \text{ m}$
- sill height: 3 m
- mean sea level = $z' + 6.1 \text{ m}$

- tidal amplitude = 0.60 m
- tidal period = 12.5 hours

The sluice model was run for $1 \leq m \leq 7$. When opening only one gate, the target lake level could not be reached even when lifting the gate completely. When using two gates, the target level is reached, but the modular flow limit is exceeded for the greatest part of the discharge period. This results in unwanted transitions to intermediate and free flow with fluctuating discharges that are hard to control. For $3 \leq m \leq 7$ strictly submerged flow exists and the target is met. Therefore, only these configurations are modelled further. The plotted water levels (Figure 12) show that the lake level fluctuates in a controlled way and is nearly identical for the scenarios with and without discharge control.

In Figure 13, the gate openings and achieved gate discharges in time are plotted for one tidal period for the situations with three or seven gates opened during the discharge event. Intermediate numbers of operated gates ($4 \leq m \leq 6$) lie between the shown curves for $m = 3$ and $m = 7$, but are not plotted for clarity. It can be seen that constant gate openings give discharges that vary in time following the time-dependent hydraulic head difference. In the PID-controlled scenario, the gate opening is automatically operated in such a way that the discharge stabilizes quickly after the start.

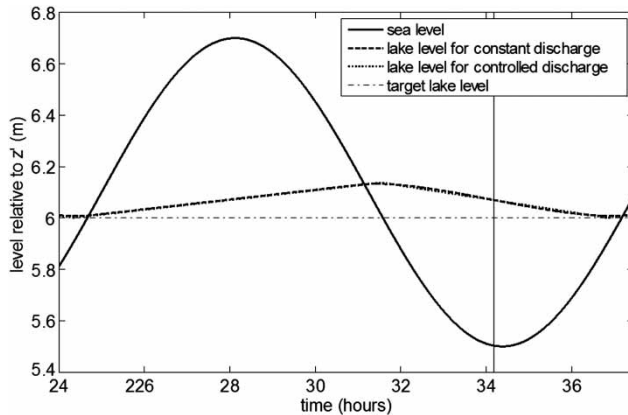


Figure 12 | Results of sluice model for $3 \leq m \leq 7$: sea and lake level for gate operation scenario with and without PID-controlled discharge. Vertical line indicates moment of maximum head difference.

In this multi-scale modelling approach, averaged values from the discharge model are used to improve discharge predictions at system scale. However, instantaneous discharges and gate openings computed in both models inevitably differ. The largest discrepancies are around 10%. This could be improved by examining different update methods, at the cost of longer computation time.

Three configurations are selected for evaluation by free-surface simulations. These cases are marked in Figure 13 as runs I, II and III. Runs I and III represent extremes: a constant gate opening with only three gates in use (high Q) and a controlled opening with all seven gates in use (low Q). All three runs are at the time of maximum head difference. In real-life practice, more cases could be selected for

simulation depending on specific interests and available computing power.

Results of CFD simulations

To simulate the two selected runs I and II within the validated range, the levels and opening are scaled down with length scale 1:10, see Table 1.

The near-gate flow velocities, pressures, TKE and dissipation are simulated. Figure 10 shows a plot of the simulated flow field of run II (at length scale 1:10) by indicating \vec{u} .

The simulated free surface as expected sinks in the region directly downstream of the gate (solid line in Figure 10). In this case, the vena contracta is located at short distance downstream of the flow separation point. The separation between positive and negative horizontal velocities in the recirculation area is derived (dashed line in Figure 10). At a distance of around five times the downstream water level past the gate, the flow reattaches at the surface and the velocity starts to return to a more uniform profile.

Figure 14 shows plots of the pressure and TKE of run II.

In the case shown in the plots, the equilibrium state reached in the simulations is fully steady. Pressure gradients are mild; the pressure returns smoothly to a hydrostatic shape as the streamlines become parallel downstream. The TKE reaches a maximum in the middle of the water column at about two times the downstream water depth

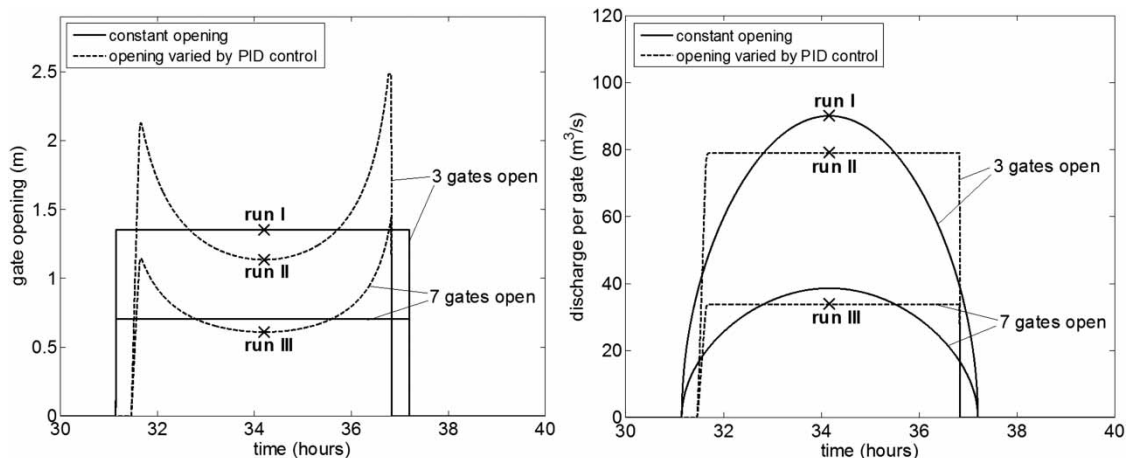


Figure 13 | Results of sluice model: gate openings (left) and achieved discharges per gate (right).

Table 1 | Values of selected CFD runs

Run	Gate configuration	Length scale	Water levels (m)				Gate opening a (m)	Total discharge Q_{tot} (m ³ /s)	Discharge per gate Q_i (m ³ /s)	Discharge per gate per unit width q_i (m ² /s)
			h_0	h_1	h_3	h_4				
I	$m = 3$, constant opening	1:1	3.07	2.93	2.38	2.50	1.30	270	90.1	4.00
		1:10	0.307	0.293	0.238	0.250	0.130	0.855	0.285	0.127
II	$m = 3$, PID control	1:1	3.07	3.00	2.44	2.50	1.14	237	79.02	3.51
		1:10	0.307	0.300	0.244	0.250	0.114	0.750	0.250	0.111
III	$m = 7$, PID control	1:1	3.07	3.06	2.49	2.50	0.610	237	33.86	1.51
		1:10	0.307	0.306	0.249	0.250	0.0610	0.750	0.107	0.0476

■ Input values for CFD runs.
All water levels h_i are relative to $z = 0$.

past the gate. Run I has a steeper surface behind the gate than run II (shown in Figures 10 and 14) and higher TKE levels, while run III has the lowest TKE levels and the most level surface downstream of the gate.

Results of flow analysis

The output of the CFD free-surface model is used for computing the values of the three flow parameters that were discussed in an earlier section, see Table 2.

Table 2 shows that the contraction coefficients do not differ much, which is expected for similar gate types. The velocity in the control section U_{vc} is highest for the situation with highest discharge per gate (run I) and lowest for the situation with smallest discharge per gate (run III). The same holds for the Froude number. This matches

observations from the free surface curvatures of the final solution of the transient simulations.

The flow impact on the bed protection material is estimated by computing Ψ for two different α for the selected runs. The whole water depth d is used for averaging the square of the maximum local velocity term $(\bar{U} + \alpha\sqrt{k})^2$. The results are plotted in Figure 15.

The plot shows that run I (three gates with constant opening) has the strongest flow impact on the bed material of the three runs irrespective of the choice for α . The Ψ -values of run II show that controlling the discharge without opening more gates already gives a lower flow impact on the bed. Run III (seven gates with controlled discharge) has the lowest flow impact. All runs reach their maximum flow impact on the bed around the same (limited) distance downstream of the gate. For all runs the general shape of the curves

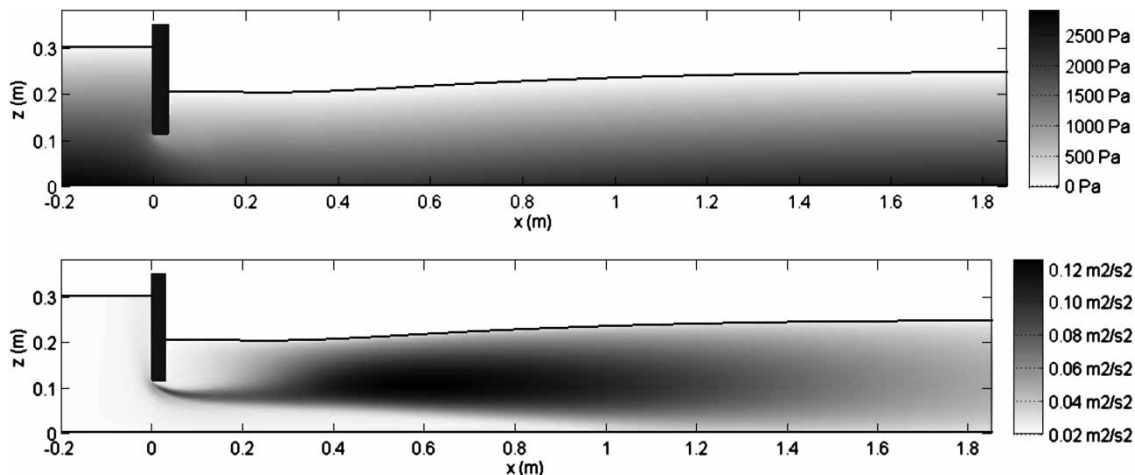
**Figure 14** | Pressure p in Pa (above) and turbulent kinetic energy k (TKE) in m²/s² (below) of run II.

Table 2 | Computed flow parameters derived from CFD model results

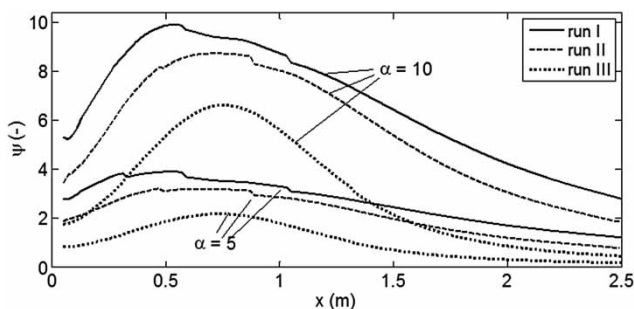
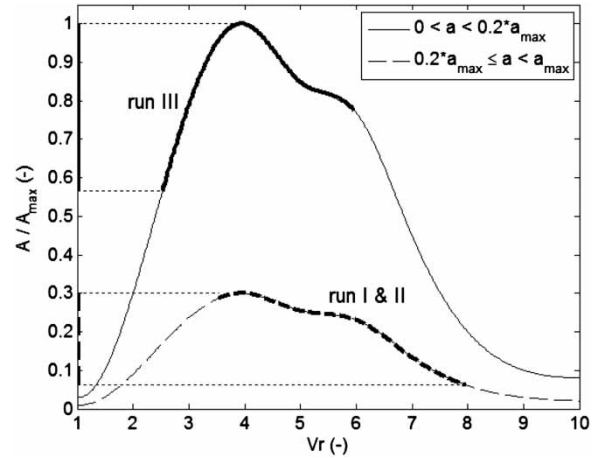
Run	C_c (-)	U_{vc} (m/s)	Fr (-)
I	0.88	3.56	0.83
II	0.86	3.50	0.78
III	0.84	2.74	0.57

is quite similar for both values of α , indicating that turbulence is dominant over mean velocity for the flow impact.

Overall, the values of the bed stability parameter are somewhat low compared to previous numerical investigations by Erdbrink & Jongeling (2008) and Erdbrink (2009), which could be attributed to the use of the standard k-epsilon model in this study instead of the RNG k-epsilon turbulence model used in the two mentioned studies. Choosing higher α values could compensate the lower TKE. For practical application one should fix α after calibration in experimental investigations and define a threshold value for Ψ that should not be exceeded during operation and that can be used as a fitness measure for different flow scenarios.

Turning to the assessment of gate vibrations, it is calculated that for an assumed range of structural response frequencies of 2–5 Hz (typical values for large hydraulic gates), the reduced velocity number Vr lies in the range 3.5–8.5 for runs I and II and in the range 2.5–6 for run III. For illustration purposes, a response curve is devised, see Figure 16, since a full evaluation is laborious (e.g. Billeter & Staubli 2000). Projection of the Vr -values onto the response curve give resulting vibration amplitudes.

Two different response curves are used in Figure 16. The most significant excitation of cross-flow vibrations of a flat-

**Figure 15** | Computed values of bed stability parameter Ψ downstream of the gate for two different values of turbulence impact parameter α . Runs I, II and III are shown.**Figure 16** | Gate vibration response for runs I–III giving relative amplitude A / A_{\max} as a function of reduced velocity Vr . Fictitious response curves are used to illustrate the method. Two regions of gate openings are distinguished.

bottom gate occurs at small gate openings, therefore higher amplitudes are expected for run III. In this fictitious instance, the computed Vr -ranges indeed give higher relative amplitudes for run II than for the other two runs. As with the bed protection assessment, the definition of a literature-based threshold level would be a logical addition for real applications.

Based on the discussed modelling results and flow analysis, it may be decided to implement the discharge scenario of run II, because it leads to acceptable vibration levels and gives a lower impact on the bed material than run I – while still ensuring sufficient discharge volume to reach the target lake level.

RECOMMENDATIONS

As a main recommendation, we propose to apply this modelling process in a case study of existing barrier structures such as Haringvliet, Oosterschelde, Maeslantkering in The Netherlands or the Saint Petersburg barrier in Russia. This research should find a natural place within on-going work on system-scale modelling for water level prediction used in decision support for hydraulic structures (Boukhanovsky & Ivanov 2012).

Specific gate uses are to be simulated and evaluated. For the last two barriers just mentioned, the operational modelling system will be mostly aimed at widening the window of

operation. The introduced methods can also be adapted for weirs in rivers. Coupling the presented models with a mid-field or far-field model of regional scale would enable an operational impact assessment for water management issues such as salt water intrusion.

The inclusion of measurement data (from field sensors or laboratory tests) is necessary for the calibration of empirical parameters (such as entrance and exit losses), for the process of model validation and for providing actual model input (water levels). Experiences from the field of hydroinformatics should be added to the present research to make the extension towards data-driven modelling components. The link with data assimilation that is to be accommodated by the higher-level models is obvious.

A longstanding issue in the engineering practice of detailed hydrodynamics is turbulence modelling. The right balance between accuracy and computational costs needs to be found for specific applications. Again, smart use of measurement data for numerical validation and calibration could be the key. It is furthermore expected that intermediate and free flow conditions where hydraulic jumps occur away from the gate, including the Venturi flow type, can be modelled more universally using other numerical methods such as Phase Field or Volume of Fluid. If needed, the model can thus be extended to account for dynamic effects directly related to opening and closing actions of the gates. Active setpoint ramping of the PID-control using feed-forward model predictions is another recommendation related to this.

CONCLUSIONS AND FUTURE WORK

The purpose of the current study was to set up physics-based modelling methods for a flow-centred operation of gates of hydraulic structures. The described case of a multi-gated outlet barrier sluice has shown how discharge estimates and free-surface simulations can aid in deciding on optimal gate configuration and opening scenarios.

The application of a PID-controller to achieve a more constant discharge during changing head differences emerged as a feasible addition to traditional structure operation. Prediction of gate discharge coefficients is a central issue in determining appropriate gate openings in the

control process. A combination of elementary equations and empirical relations was used for this. The increase in computational power over the years now enables solving these flow equations in quick assessment procedures during operation.

Free-surface CFD simulations of the turbulent flow past an underflow gate revealed the effects of local lowering of the surface on flow velocities and TKE levels. Time-dependent FEM simulations with a moving mesh technique were found to give stable solutions of the free-surface under submerged conditions. From a series of validation runs it is concluded that the free-surface model yields discharge values for a range of gate openings and submergence levels within an acceptable accuracy of experimental values.

Among the flow analysis possibilities based on output from the free-surface model is computation of the Froude number, the reduced velocity parameter for estimating gate vibrations and a stability parameter for granular bed protection. The numerical example of the discharge sluice has proved the feasibility of combining discharge estimates with free-surface simulations for deriving operational decisions. For the particular case treated in this paper, it was found that lower TKE levels of the PID-controlled discharge scenarios contribute significantly to reducing the flow attack on the bed protection. Additionally, the model showed the influence of the number of opened gates on the flow properties.

The practical benefits of including near-field flow modelling in gate control systems seem clear. It will enable more sophisticated water reservoir management in everyday operation with respect to issues such as salt water intrusion, fish migration and possibly saving energy. In extraordinary situations, model results can help maintain safe gate usage and prevent gate vibrations, washing away of bed protection and the development of scour holes around the structure.

Limitations of the followed modelling approach need to be addressed in follow-up studies. Additional calibrations are necessary: PID-control optimization to obtain the desired discharge more precisely, discharge and loss coefficients in the flow equations and turbulence model parameters. Next to this, improvements to the free-surface model should broaden the range of applicability so that steeper surface disruptions and hydraulic jumps as found in free flows can be captured as well.

The physics-based model of this study is logically complemented by data-driven techniques in future studies. It is believed that hydroinformatics provides the required tools for this. Use of sensor data from real-life structures and coupling to system-scale water level prediction models are seen as next steps. Moreover, it should be investigated how operational decisions should be derived when taking into account the various criteria and flow constraints.

ACKNOWLEDGEMENTS

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