FLAP GATE PERFORMANCE IN HYDRAULIC MODELS

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ABSTRACT

Flap gates are common at waterway confluences where flow in a particular direction (usually upstream flow) is to be prevented. As such structures are relatively common in waterways that are subject to flood-related constraints, and since hydraulic modelling is often used in such situations to aid with decision making, it is often important that the performance of flap gates is adequately represented in such models. Many models allow the user to permit flow only in one direction, and it is this representation of what in reality might be a flap gate that requires clear understanding.

The degree of opening of a flap gate is dependent on the head difference across the structure. For a small head difference the opening is severely constrained, and the flap gate opens wider with increasing head difference. Full open flow is seldom achieved with older flap gates, since these are generally heavy and do not lift fully even under high velocity conditions. More modern counter-weighted flap gates open more under small head differences, and are arguably more efficient.

There have been studies undertaken to evaluate the head loss across flap gates for a range of different flow rates, and it is this type of accuracy in performance that often needs to be incorporated into hydraulic models. Simply permitting flow in only one direction implies a fully open structure regardless of head difference across it, where in reality the degree of opening (and hence head loss) at such a structure is dependent on head difference across it. This is further complicated when water level either side of the structure is below obvert level, and full flow does not occur.

This paper examines methods for adequately representing flap-gated outlets in hydraulic models.

KEYWORDS

Flap Gate, Hydraulic Model, Head Loss

PRESENTER PROFILE

Mark Pennington MIPENZ CPEng is a Civil Engineer with some 14 years of engineering experience, the majority of which has been spent in hydrological and hydraulic investigations and analyses. He has a Master's degree in hydraulics and his main focus in the last few years has been in urban stormwater management and in flood management for river systems. Mark is a committee member of the IPENZ/WaterNZ Rivers Group and of the WaterNZ Modelling Special Interest Group.

1 INTRODUCTION

A Flap Gate is the name frequently used to describe a structure in a waterway that permits flow in only one direction. These are also referred to as "Floodgates" or "Tide gates", and are commonly used at the ends of piped outlets from either pumped or gravity systems to prevent backflows. Sometimes these are also used to restrict the movement of animals in open waterways, where a large opening for a waterway would otherwise form an effective hole in a fence under low flow conditions.

"Upstream Water Control Gates" are similar to flap gates as defined above, in that their geometries are based around similar principles. However, as their name suggests such gates are more frequently used to control upstream water level within a small range, often a requirement for the effective functioning of irrigation canals. These gates allow increasing discharge to downstream as upstream head increases, thereby controlling upstream water level. To potentially add confusion, “upstream water control gates” are sometimes, themselves, referred to as “flap gates”, and it needs to be made clear that these gates are not the subject of this paper, even though much of the discussion would apply equally to both gate structures.

Flap gates vary widely in shape and opening characteristics, with some being hinged vertically while others are hinged horizontally. Yet others have counter-weights to assist opening; some are constructed of steel and some of fibreglass reinforced polyester materials. Generally gates are either rectangular or circular in shape, with circular ones frequently hinged horizontally and rectangular gates hinged vertically.

In some cases the performance of flap gates can be crucial in operation of a land drainage system. The hydraulic performance of the flap gate can influence upstream water levels and discharge to downstream, in addition to serving the basic function of prevention of backflow.

While maintenance of flap gates is essential to ensure their correct function, this paper focuses on the performance of properly maintained structures and does not dwell on practical maintenance issues, most of which would potentially have a larger effect on upstream water levels than the actual hydraulic performance of the gates. Furthermore, this paper examines the aspects of gate opening that are of significance to modellers seeking to accurately represent the performance of such structures in hydraulic modelling situations.

2 TYPES OF FLAP GATES

There are several different flap gate configurations commonly in use in New Zealand. In stormwater situations the circular pin-hinged type of gate is certainly widely used, as are flexible gates. These are shown in Figure 1.

Figure 1: Pin-hinged circular (left) and flexible (right) flap gates

Flexible gates are generally made of rubber with some steel reinforcing, and enable a good seal under back-pressure conditions while requiring little positive head to effect opening.

In addition to the above, double-hinged circular gates and rectangular box-culvert type gates are also encountered. An example of a double hinged flap gate is shown in Figure 2. Double-hinged gates have a more complex opening mechanism than single-hinged gates, potentially allowing for greater discharge with lower head loss.

Rectangular box culvert flap gates are sometimes hinged vertically, meaning that they can open and close in a similar fashion to how a door operates. Because of this arrangement gate weight does not act directly against the opening moment, but rather contributes to hinge friction. Difficulties associated with leakage against negative hydraulic grade is frequently encountered with such gates.

Figure 2: Double-hinged circular flap gates

3 HYDRAULIC PERFORMANCE

3.1 THEORY

As stated in the Introduction above, a flap gate's purpose is generally to prevent back flow. If it is defined that flow is positive in a downstream direction, then under conditions where a positive hydraulic grade exists (i.e. upstream water level exceeds downstream water level) then the gate is able to open. The degree of opening is dependent on several factors, the most significant of which are head difference and flow velocity across the gate and gate weight.

A schematic geometry sketch for a single-hinged circular flap gate is shown in Figure 3. In this H_1 and H_2 are upstream and downstream heads respectively, with the difference in head across the gate given by • H. The angle through which the gate is able to open, •, is derived from balancing the forces on the gate, these being the resolved components of the forces **F** (hydrostatic thrust on gate) and **W** (weight of gate). It can be seen that as **W** has no horizontal component, when **F** > 0 (i.e. • H > 0) there must be a degree of opening (i.e. • > 0). To a minor degree there are other forces such as hinge friction and adhesion that may come into play that could reduce the resultant opening couple on the gate. Similarly, it can also be seen that as **F** has no vertical component to balance against **W**, it is theoretically impossible that • = 90•, which would imply a completely open gate with no cross sectional restriction.

The opening couple on the gate is complex to compute analytically. Many references show resolution of forces for rectangular gates, but a literature review found no easily available analyses for circular gates. For example, Raemy and Hager (1997) examined the pressure distribution on a rectangular flap gate (actually an Upstream Water Control gate as defined in this paper). They were able to determine that a linear pressure distribution only exists at the gate for zero flow due to water exiting under the gate being at atmospheric pressure (assuming un-drowned downstream conditions). This implies that the difference between static force (when gate closed) and the actual force (when gate open) increases with the degree of opening of the gate.

In a subsequent paper, Raemy and Hager (1998) proposed to estimate **F** by assuming a linear relationship with gate opening, •. The momentum equation was used to estimate **F**, and their analysis was confirmed by their experimental results. However, it is difficult to apply these results to other gates as the analysis does not give the pressure distribution on the gate, and hence the opening couple on the gate cannot be determined. In addition, energy losses would no doubt be incurred as flow passes through a partially open gate, and it is not clear whether or not this has been fully taken into account.

In addition to the force balance described above, there is a further component of back-pressure applied when a flap gate does not discharge to free outfall conditions. For small • H this back-pressure effect would be more noticeable, particularly if the degree of submergence downstream is high. In spite of this, it takes relatively small • H to cause the gate to open. Replogle and Wahlin (2003) quote an example of an iron gate about 12.5 mm thick, with a mass of 15 kg covering the end of a 380 mm diameter pipe at end angle of 1:10 as requiring a head difference, • H, of just 39 mm to initiate opening. Note that as explained above, a gate that is horizontally hinged would theoretically begin to open with • H > 0, since there would be no significant opposing forces to resist opening. A better seal with some opposing force due to the weight of the gate is able to be attained by offsetting the hinge from the plane of the gate, such as that shown in Figure 1.

When flow through a flap gate is large then pressure forces become less dominant, with the open gate being supported by jet forces caused by deflection of the jet. In this case it can be seen that the mass of the gate would be a significant factor in determining the degree of opening and hence the likely head loss and/or discharge at the structure.

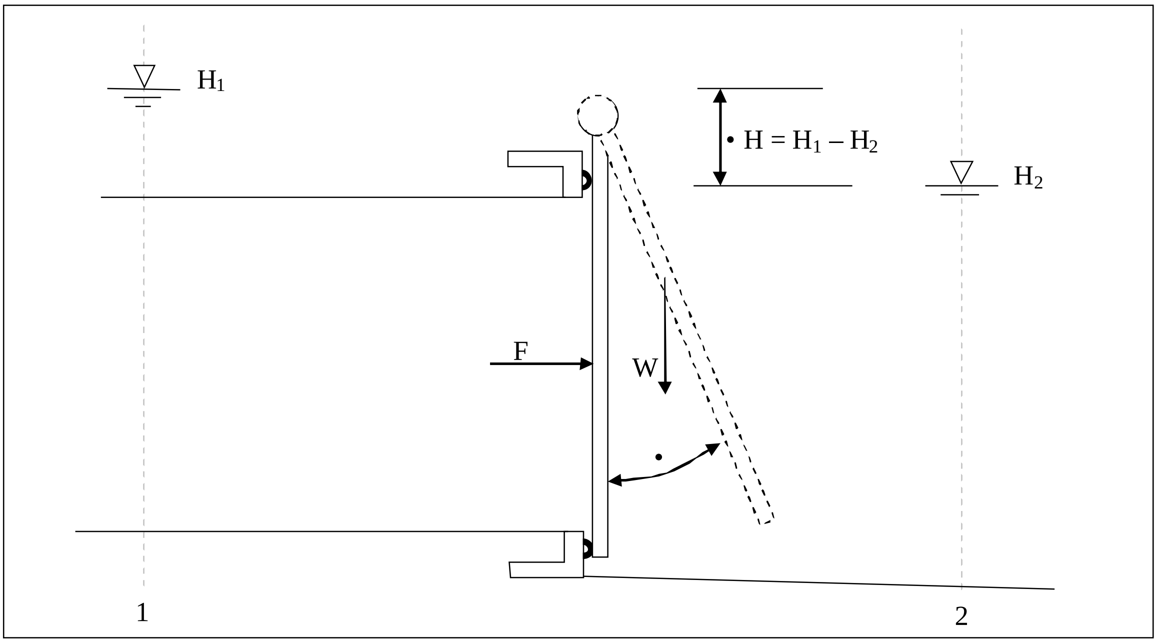

Figure 3: Schematic Section through Single-Hinged Flap Gate

The discharge through the flap gate structure is the product of area and velocity. In the piped section upstream of the gate, flow area is the cross sectional area of the pipe and velocity is the average velocity over the section. As flow passes through the flap gate itself, flow area is reduced from pipe cross sectional area, and is dependent on, amongst other things, the degree of opening of the gate, •. As flow area is reduced, the principle of continuity requires that velocity must increase. This is important for any energy loss function that employs velocity head, defined as $v^2/2g$ (where v is velocity and g is acceleration due to gravity).

In examination of flow through a flap gate structure, both energy and momentum equations should always hold true. The energy equation, which relates to dissipation of kinetic energy, will hold true as long as all “losses” are accounted for between two sections. Similarly the momentum equation will always hold true provided that proper allowance is made for all forces acting. The Bernoulli expression, which relates to energy losses, is applied between two sections (say section 1 and section 2 in Figure 3), and allows calculation of pressure difference for a given discharge. This expression does not, however, allow for evaluation of the net thrust **F** that is applied *between* these sections, and this is best determined using momentum equation principles.

Energy losses are usually accounted for as a factor of velocity head, $k.\ v^2/2g$. With flow area, A, being difficult to determine over the opening range of a gate, velocity v is unknown for a given discharge and hence velocity head cannot be computed.

This presents a potentially complex problem to modellers, who are generally allowed freedom to select *either* energy *or* momentum solutions, and it points to an alternative approach being required.

No doubt the complex mathematics could be done for resolution of forces acting and for changes in flow area around a partially open gate, but this is outside the scope of this paper and may be of little practical use to modellers. Furthermore, this type of analysis would be specific to a single gate and would not necessarily apply over other gates. Of importance to modellers is how such structures may be accurately represented in a model, such that model performance is able to closely match actual performance.

Litrico *et al* (2005) analysed the opening mechanisms for an upstream water control gate, and presented a mathematical model for a specific gate geometry, namely a *Begemann Gate*[1]. This model is stated as being suitable for inclusion in a classical hydraulic simulation model that solves the *St-Venant* equations. This model allows for computation of upstream water elevation, H_1, and gate opening angle, •, for any given discharge, Q. In their paper Litrico *et al* compare results of the derived mathematical model with experimental data, and show a good fit. Fortunately they also compare these two sets of results with those that would be obtained using a standard weir-flow relationship, which effectively assumes that the gate is fully open under even the smallest positive hydraulic grade. This is fortunate in that this is a simplified assumption that is frequently applied in such situations, and the differences are relevant and should be understood. One of the relevant sets of results is plotted in Figure 4.

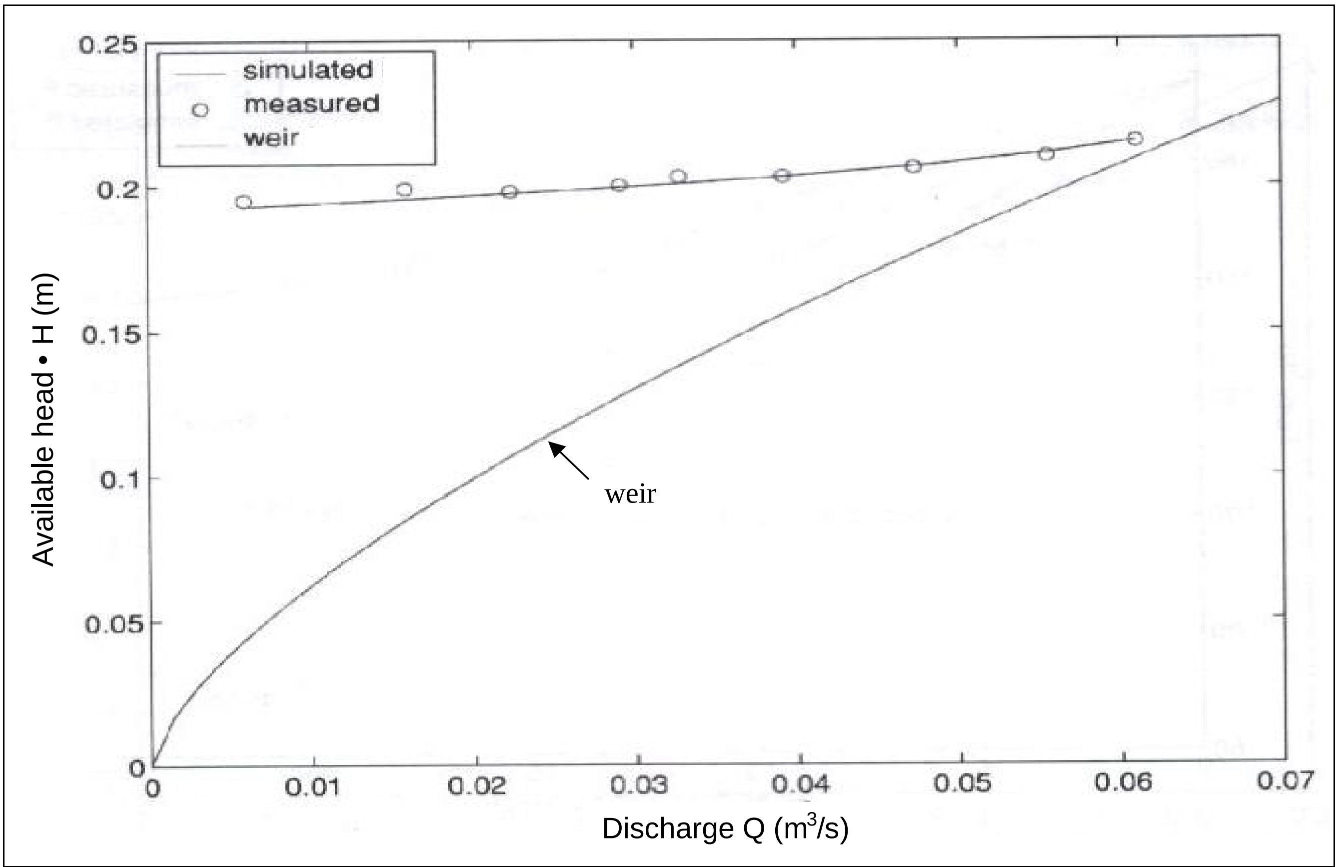

Figure 4: Simulated and Measured data plotted against weir-discharge relationship (Litrico et al, 2005)

What can be clearly seen in this figure is the good fit between simulated and measured data obtained in this study. Also immediately evident is the discrepancy between the measured data and the performance that could be expected using a weir-discharge relationship, particularly at lower values of discharge, Q. As this study focussed on upstream water control gates with free outfall conditions, the comparison with the weir-discharge relationship is a valid one. However, since the mathematical model developed applies only to the specific geometry of a *Begemann Gate*, it may have its limitations for wider applications for flap gate modelling in its current form. No

[1] The Begemann Gate referred to is an automatic upstream water level control gate that controls upstream level close to a reference level across a range of flows, using a counter-weight to compensate for hydraulic pressure. Horizontal hinge.

doubt this would make a valuable starting point for an interesting investigation into similar models for other specific types of flap gates.

The difference in performance highlighted in Figure 4, particularly at low discharge, is potentially of relevance to modellers.

3.2 PRODUCT LITERATURE

An extensive search of available literature was conducted, with a view to finding data on the hydraulic performance of different flap gates under differing head conditions. Little suitable data were found.

In one reference a relationship between head loss and velocity is given, having been derived from a series of tests conducted at the (then) State University of Iowa. This relationship is plotted in Figure 5, and applies only to a specific type of gate, known as a Calco Gate. It is understood that such gates are made of steel and are double-hinged. It is not clear from this reference whether or not downstream conditions are taken into account, and it has to be assumed that free outfall conditions were applied as otherwise additional performance data would be required to cover a range of possible downstream conditions. As such these data are of use in capacity analyses, but are of limited use in a model used to simulate unsteady flow over a flood or tidal cycle.

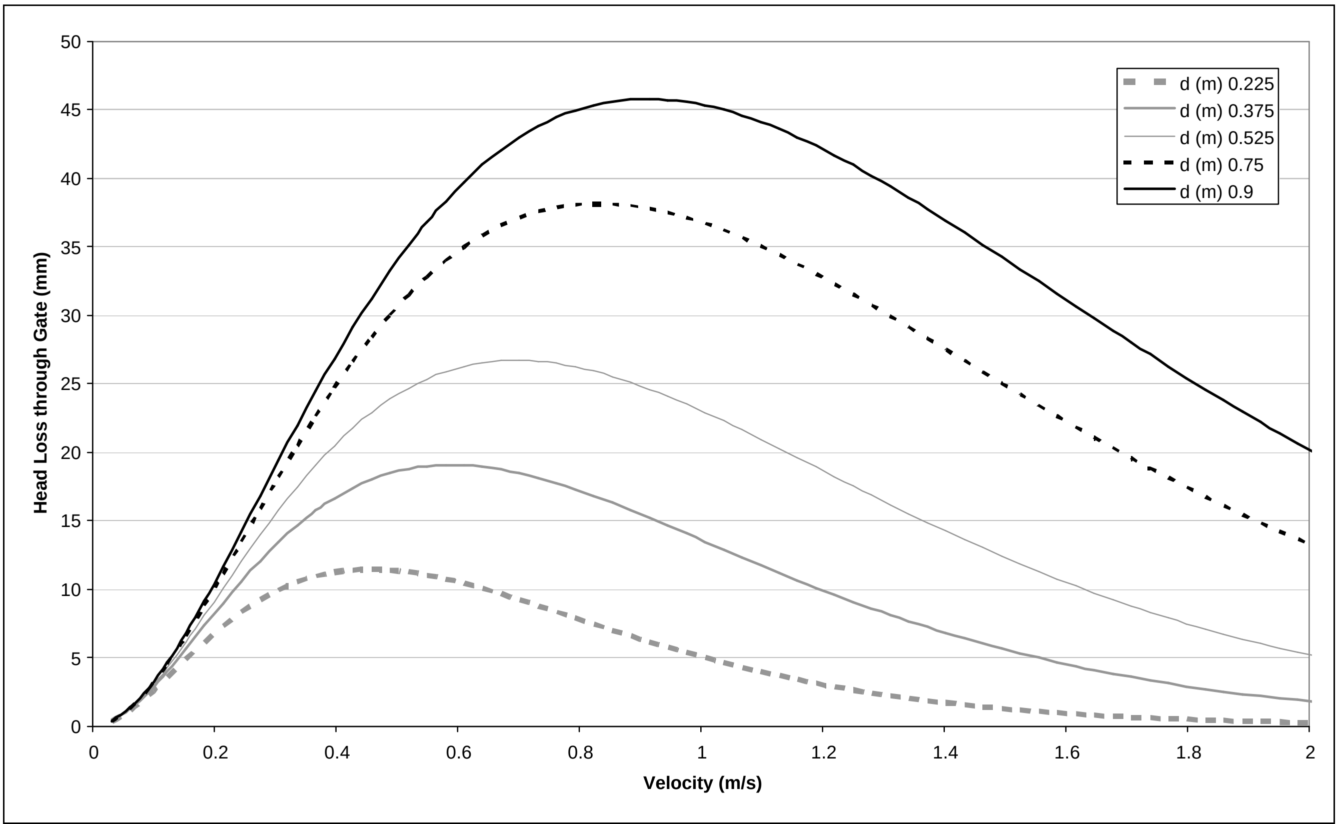

Figure 5: Head Loss through Calco Gates of different diameter (Hydro Gate Ltd, Co.)

This shows that head loss increases with velocity to a maximum and then tapers off as velocity is increased further. It is understood that in the tests conducted the velocity applies to average pipe velocity, obtained by division of discharge by cross sectional area. This is not necessarily the velocity of the jet as it accelerates past a partially-open gate. The reason for the rise-and-fall performance displayed in Figure 5 is likely to be attributable to degree of gate opening over the range in velocity. For low velocity, the gate is open less widely and higher losses are incurred, and once velocity increases sufficiently the gate is able to be held open more widely with lower losses.

Another reference gives head loss in terms of equivalent pipe length, over a range in flap gate diameter. This shows a linear relationship between equivalent pipe length for head loss computation and flap gate diameter, and since pipe head loss is directly proportional to length, implies a linear relationship between head loss and flap gate diameter. The study does not reference head losses over a range in discharge, but appears to focus on a design capacity as the discharge for which the given head loss would apply.

This leaves little for modellers to use in accurate simulation of flap gates, particularly since models generally run in unsteady mode. Detailed balancing of forces and resolution of relationships between gate weight, flow velocity and head loss have been shown to be complex and difficult to solve. Product literature in which the results of physical tests may be sought seem difficult to come across, and there is a general paucity of detailed information on these structures.

When considering head loss and gate opening mechanisms, it is likely that a relationship exists whereby degree of gate opening, •, is proportional, in the first instance, to head difference across the gate, • H. Once gate opening has been initiated by a positive static head difference across the gate, the degree of opening is likely to be dependent on approach flow velocity, v, in addition to • H. A balance would be reached given constant upstream and downstream head and gate opening, and hence discharge.

4 MODELLING APPROACH

4.1 SIMPLE APPROACH

Many hydraulic modelling software packages allow for prevention of backflow through structures. This is often effected by selection of an option that disallows negative flow. Using this approach, as soon as a negative hydraulic grade is encountered at the subject structure, discharge through the structure is set to zero, so preventing upstream (or negative) flow.

This is relatively simple and often of suitable accuracy, and many modellers elect to simulate flap gates in models using exactly this approach. However, a corollary to setting discharge to zero at the onset of a negative hydraulic grade is that full cross-sectional flow area is made available for even the smallest positive hydraulic grade. In this way the flap gate is simulated more like a switch, which is either “on” for positive flow or “off” for negative flow. This is sketched in Figure 6 below, where the gate is shown completely shut for • H • 0, and completely open for • H > 0.

From the discussion above, the geometry of the opening flap gate under increasing head differential is complex to analyse. For small • H it would be expected that Q would be over-estimated by models using the “prevention of negative flow switch”, where in reality small • H results in a small degree of opening and hence greater losses through the structure. There are many situations where this approximation would not matter at all. However there are also situations where this may be critical. An example of this could be a large ponding area discharging to a tidally controlled outlet. At times of high tide, outflow from the ponding area is prevented and upstream water level would continue to rise as inflow is added. As soon as downstream tide level drops below upstream pond level, a model using the “switch” approach would allow full flow to downstream where in reality a period of partially-open gate would severely restrict the outflow.

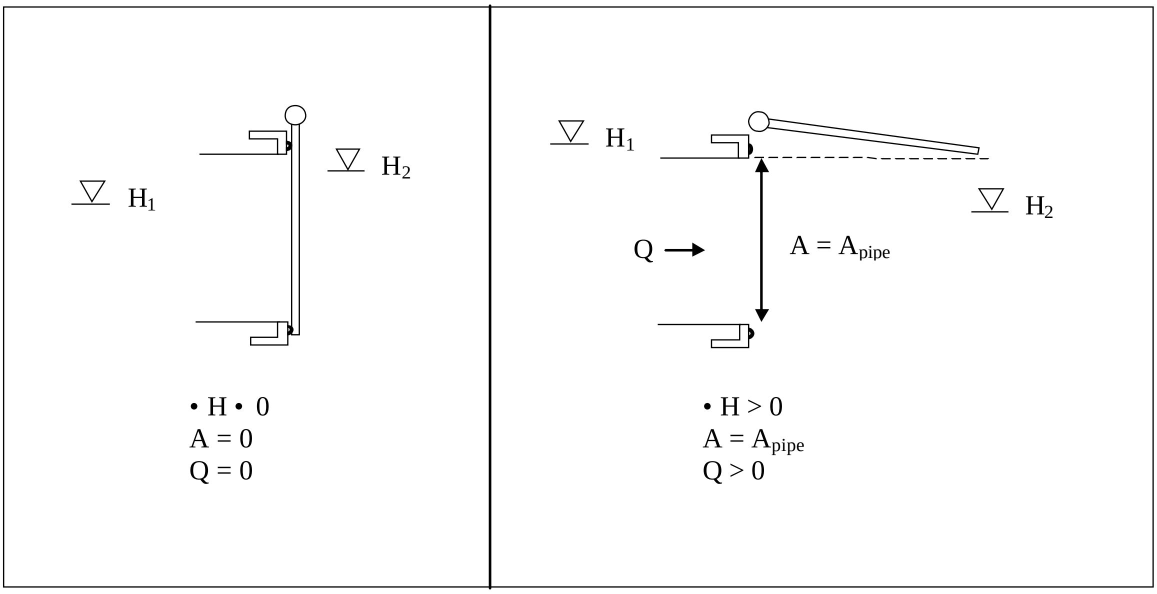

Figure 6: Flow configurations allowed by "prevention of negative flow switch" in a model

Once the tide comes in again, the outflow is once again stopped, and the ponding area behind the gate continues to fill. However, since a model using the "switch" approach would allow for greater outflow than would occur in reality, the modelled level in the upstream pond at the point in time when outflow is again prevented would be lower than that which would occur in reality. Should the peak of a flood arrive during the time at which gates are closed, then given a slightly higher starting water level it would be anticipated that the real flood level would rise to a level in excess of that predicted by this model.

Should the duration of a flood span several tidal cycles, without complete emptying of the upstream ponding area between periods of gate closure, then this effect may be amplified with time, widening the gap between modelled and measured upstream pond levels.

4.2 SUGGESTED APPROACH

As outlined in Section 3.2 above, some flap gate suppliers are able to provide hydraulic information for various products. Such information does seem sparse in the wider industry, but in some cases the data may be able to be applied more widely.

At the very least the head losses quoted by manufacturers under free outfall conditions should be ensured in any hydraulic model. Even if the actual situation is such that free outfall conditions never exist, it may be prudent for the modeller to calibrate gate performance under artificially-created free outfall conditions before applying more realistic downstream boundary conditions.

By reference to Figure 4 it is reasonable to anticipate a difference in flap gate performance should a simplified modelling approach be applied. Some correction of this performance will therefore be required if critical model results are heavily influenced by modelled flap gate performance.

5 SAMPLE CALCULATION

A simple model has been constructed and run to display differences in performance of a flap gate under a range of conditions. In order to be able to make use of performance data found from the literature survey (the most useful being that shown in Figure 4), the model was set up such that the gate structure exhibits the "weir" flow relationship under free outfall conditions. The model comprises a floodplain reach, with cross sections 100 m wide, connected via this weir structure to an outlet reach. A theoretical flood hydrograph is applied as inflow boundary at the upstream end of the floodplain reach, and the downstream boundary at the outlet reach is a tidally-fluctuating water level. Invert level at the structure is set to 0.0 m with tide fluctuating from -1.45 m to +1.45 m.

The model was run using the "negative flow prevention switch" over several tidal cycles, with the arbitrarily derived flood inflow hydrograph as upstream boundary. In this way outflow is allowed to occur as soon as upstream head exceeds downstream, in accordance with the "weir" relationship plotted in Figure 4. This was done using a simple Q-H relationship, with outflow Q being dependent on upstream Head H_1.

In order to account for actual performance of the flap gate, it is necessary to examine the head difference • H across the structure at every computational timestep in the model. This option is not available in many modelling packages, and as a result a mathematical model using *Matlab* was set up. This was set up to allow outflow rate through the structure to be dependent on • H (rather than on H_1 as can be done using a simple Q-H relationship). The modelled discharge is dependent on the head which is taken to be the lesser of the upstream stage and the difference between the upstream and downstream (tidal) stage. In this way the actual performance as displayed by the experimental data from Figure 4 was built into the model.

The model results are plotted in Figure 7. These show upstream water level for the two scenarios. Immediately obvious from the first plot is that peak water level upstream of the structure is under-estimated using the "negative flow switch" (or "traditional" approach). Furthermore, water level is able to drain down far more during low tide cycles than would actually be the case in this example. This may have an effect on design level of service or design flood levels in such an area.

In the second plot in Figure 7 it is evident that peak discharge to downstream is higher under the corrected flap gate scenario than would otherwise have been anticipated, particularly for tidal cycles after the simulated flood event. This is not because of any larger opening being formed but merely because upstream water level is higher when this flap gate configuration is used (due to more constrained outflow over each cycle). This may result in effects to do with downstream erosion that are not able to be easily identified via a more common modelling approach.

The third plot in Figure 7 shows volume within the upstream floodplain, and the time series of this follows the same trend as that shown by the upstream water level plots. It is plain to see that a subsequent flood would be likely to demonstrate an even larger margin of error.

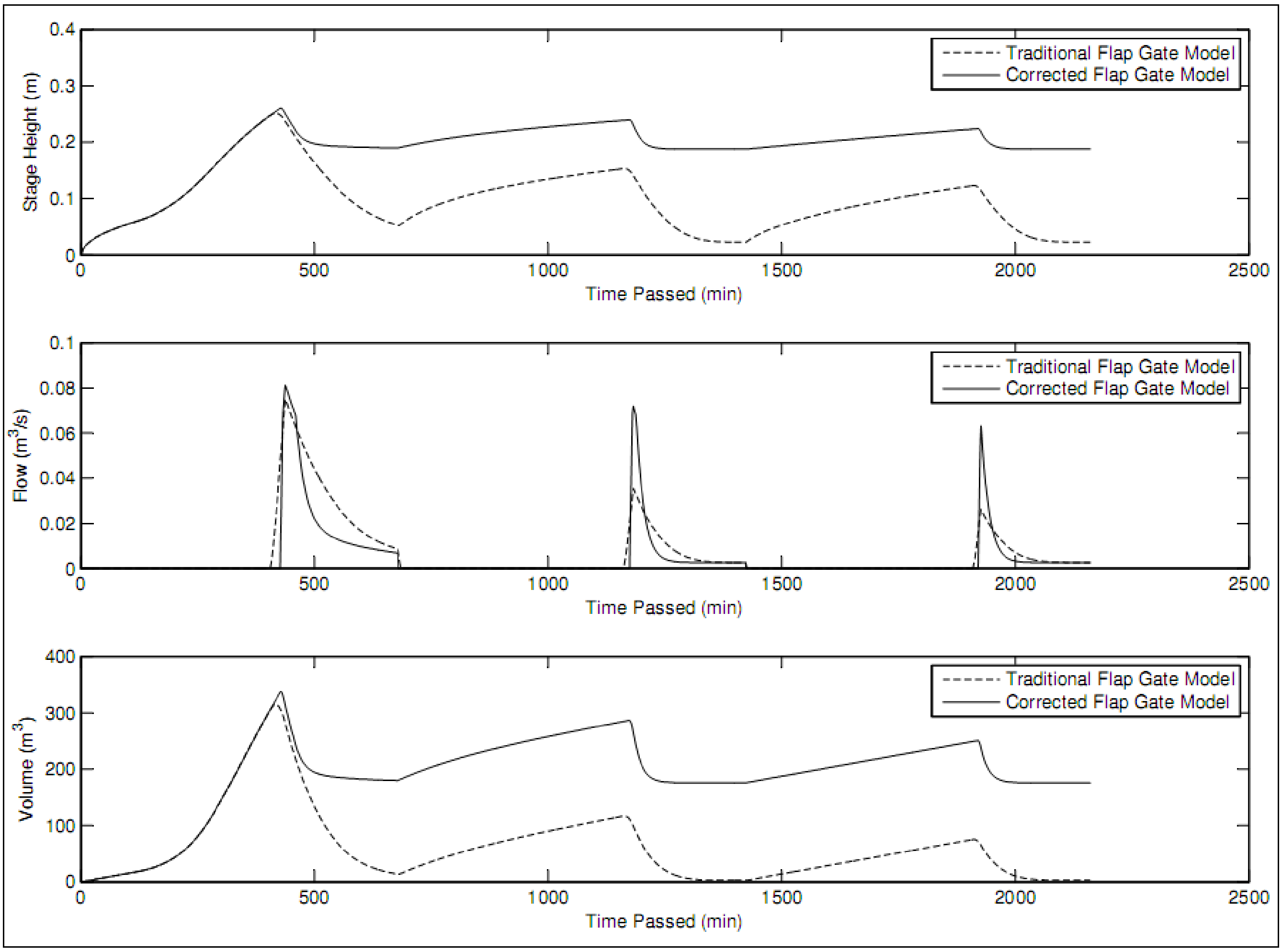

Figure 7: Water Level Results in Floodplain

It is difficult to specify modified Q-H relationships in a model since most software packages look to relate upstream head to discharge when specifying Q-H relations, ignoring downstream head. Of relevance to flap gates is the head difference across the structure, as opposed to absolute upstream head. It is of little use to specify the relationship between upstream head and discharge, as in certain conditions downstream head may exceed that upstream, in which case a real flap gate will allow zero flow.

It has been shown that in the example illustrated above, it is possible to simulate the actual flap gate performance based on experimental data, and in this case this resulted in notable differences in performance from what could be obtained via a more “traditional” modelling approach. It is accepted that the above example is purely demonstrative, and that this does not in any way simulate a trend that may be applicable to other specific scenarios. However it does demonstrate that an element of caution is required when such structures are included in hydraulic models, and where accuracy in results at affected locations is of critical importance.

6 CONCLUSIONS

- Detailed hydraulic performance data for commercially available flap gates are not readily available.
- Flap gate opening geometry is difficult to simulate accurately in hydraulic models.
- Simplification of this opening geometry can have an effect on results able to be obtained.
- Where result accuracy is critical, specific performance data for adequate calibration for the particular flap gate in use should be applied to the hydraulic model.

- In the absence of suitable calibration data, a cautionary approach when results are critical is recommended. This cautionary approach may include sensitivity analysis.

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