Lift and Swing Gate Modelling For Dam-break Generation With A Particle-Based Method

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Lift and Swing Gate Modelling for Dam-break Generation with a Particle-Based Method

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Abstract: This study investigates two gate devices which impound a volume of water, whose sudden release generates a dam-break wave. The aim is to identify the reciprocal interaction between the gate motion and the released volume of water. For the first time, lift gates and swing gates are systematically studied and compared. The influence by the opening of the gate on the evolving bore is related to the opening time. The overall objective of this study is to provide guidance as to how fast swing and lift gates need to open to effectively limit adverse effects on the formation of the dam-break wave. For this study, numerical simulations are conducted using the Smoothed Particle Hydrodynamics (SPH) formulation. The open source code DualSPHysics is used herein (Crespo et al., 2015). A calibrated setup is used to systematically investigate a large number of setups comprising constantly-accelerated lift gates and swing gates opening with constant angular velocities. To match opening procedures of existing test facilities, the distance from the gate edges to the bottom of the flume is a quadratic function in time, anticipating electrically- or gravity-driven gate concepts. Based on the analysis of the time-history of the water surface elevation, a more detailed criterion for acceptable opening times for both, swing and lift gates is provided. A comparison with existing guidance on minimum opening times for lift gates is also provided; moreover, the updated opening criteria is valid for the entire section downstream of the gates. The use of the updated opening diagrams allows to determine minimum opening times for future experimentation.

Keywords: Dam-break, Smoothed Particle Hydrodynamics, swing gate, lift gate, bottom sill, opening time.

1. Introduction

Natural disasters such as tsunamis and failure of man-made structures such as dams have caused significant damage to infrastructure, claiming thousands of lives (Lay et al., 2005). The investigation of the effects of such disasters often involves physical modelling which aims to simulating the physical phenomena present within these disasters. Experimental dam-break modelling has increasingly become a viable tool in research of such phenomena. Consequently, various test facilities have been constructed around the world to investigate the unique hydrodynamics of dam-break waves. These processes are also simulated with numerical models to reproduce dam break events (Oertel & Bung, 2012). This study investigates two gate devices which impound a volume of water, suddenly opening to create a dam-break wave. This study aims at identifying the reciprocal interaction between the gate motion and the released volume of water. For the first time, lift gates and swing gates are systematically studied and compared. The influence of the gate on the evolving bore is related to the opening time. If the opening time is too long to entirely free the cross-sectional area, the generated wave does not properly reproduce the characteristics of a dam break. A vertically-lifting gate induces vertical shear stresses on the water at its contact with the gate, whereas a swing gate potentially accelerates the water horizontally during the opening process.

Previous research has looked into vertically-lifting gates and found maximum opening velocities of such device to be a function of the gravitational acceleration and the impoundment water depth (Lauber, 1997). The authors used a 20% inclined flume and evaluated the time-histories of the free surface elevation downstream of the gate at a fixed location. Based on visible deviations increasing with slower gate velocities, the authors concluded that a threshold velocity was found. In their research, no dependence on the distance to the gate location could be established, a parameter which in practice often plays a role. Then, Goseberg et al. (2017) investigated a dam-break initiated by opening a swing gate and revealed the importance of the opening velocity for such a gate type analogously to the lift gate. Moreover, a downstream-propagating “self-healing” effect was found which – once more – indicated a dependence on the distance downstream of the gate when assessing gate opening times.

Hence, the overall objective of this study is to provide guidance on the required opening conditions for swing and lift gates needed to limit adverse effects on the formation of the dam-break wave.
2. Methodology

2.1. Numerical Setup for Dam-break Simulations

For this study, numerical simulations were conducted using the Smoothed Particle Hydrodynamics (SPH) formulation. The open source code *DualSPHysics* was used (Crespo et al., 2015). A two-dimensional (2D) model of a flume at the University of Ottawa, Canada, which is equipped with a swing gate device, was numerically implemented, calibrated and validated against a unique set of hydraulic experiments conducted within the flume (see Goseberg et al., 2017). The model setup uses a particle spacing of three millimeters, resulting in approximately one million fluid particles. The calibrated model yielded root mean square errors of less than ±3 cm when an initial impoundment water depth of 40 cm was simulated; numerical results of the surface elevation time-histories satisfactorily reproduced the experimentally-determined data. In addition, published data for a physical model test where a lift gate was employed were used to further validate the calibrated model (data taken from Khankandi et al., 2012).

The calibrated setup was then used to conduct prognosis computations. Therefore, the setup was modified to systematically investigate a large number of constantly-accelerated lift gates and swing gates opening with constant angular velocities. This numerical wave flume shown in Figure 1 was 25 m long with the gate located 15 m upstream from the exit drain, thus leaving a 10 m long impounding reservoir. To match opening procedures of existing test facilities, the vertical distance from the gate edges to the bottom of the flume was a quadratic function in time, anticipating electrically- or gravity-driven gate concepts.

![Figure 1. Numerical test setup. Reference data is generated by applying no gate to generate instantaneous dam break.](image)

The numerical tests were conducted with initial water depth of \( d_0 = 0.30, 0.50 \) and 0.70 m. The simulated time is 2 seconds and results are saved 60 times per second. Virtual wave gauges are placed along the length of the numerical flume with a distance of 0.1 m. Friction and adhesive forces can not be considered by the model. Figure 2 shows resulting free surface elevation time-histories for the calibrated model. Artificial viscosity is used with parameter \( \alpha = 0.005 \), particle spacing of \( d_p = 0.003 \) m and a relative smoothing length of \( q = 0.57 \). Additionally, the analytical approximation of a dam-break wave developed by Ritter (1892) is visualized.

![Figure 2. Calibrated model, physical measured wave gauge time series and analytical approximation. Left for swing gate, right for lift gate (physical data taken from Khankandi et al., 2012)](image)
2.2. Error Calculation

In the following sections, a metric for discrepancies between the gate types is established. To achieve a reference data set, a base model setup was run with no gate. It represents an ideal, instantaneous dam break wave. Errors were calculated as RMSE divided by the initial water depth \( d_0 \). The values of parameter RMSE/\( d_0 \) were calculated either over time or space. Parameter \( RMSE_x/\overline{d_0} \) (note subscript \( x \)) is defined as the error over the length of the flume. Consequently, \( RMSE_x/\overline{d_0} \) can be calculated for each location and is a mean error over time. The parameter \( RMSE_x/\overline{d_0} \) was limited to when the wave reaches the end of the flume \( (T = T_{\text{overflow}}) \), to limit the influence of the discharging flume as this would be a function of the flume length, irrelevant to this examination of gate types.

\[
RMSE_x/\overline{d_0}[-] = \sqrt{\frac{\sum_{t=0}^{T_{\text{overflow}}} (WG_{\text{Ref}} - WG_{\text{Num}})^2}{T_{\text{overflow}}}}
\]

where \( t \) is the time step, \( T_{\text{overflow}} \) is the time step when the flume starts to discharge, \( WG_{\text{Ref}} \) and \( WG_{\text{Num}} \) are the time series of the wave gauges in the reference and the gate affected dam break and the subscript \( x \) indicates, that this is a spatial error, calculated for a wave gauge with the position \( x \) to the gate (see Figure 1). The temporal error \( RMSE_t/\overline{d_0} \) (note subscript \( t \)) is the error at each time step and the mean error over the length of the flume:

\[
RMSE_t/\overline{d_0}[-] = \sqrt{\frac{\sum_{x=\text{m}}^{x=15 \text{m}} (WG_{\text{Ref}} - WG_{\text{Num}})^2}{N_{\text{WG}}}}
\]

where \( x \) is the distance of each wave gauge to the gate (see Figure 1), \( WG_{\text{Ref}} \) and \( WG_{\text{Num}} \) are the water depths at each wave gauge at time step \( t \) and \( N_{\text{WG}} \) is the number of \( WG \)s. Both parameters will be extensively used to analyze the below described results of the numerical model runs.

2.3. Geometrical Definition of Gate Opening

Generally, two different gate types were investigated, lift and swing gates. The lift gate is a vertical wall which can be moved upwards rapidly, to free the cross section quasi instantaneously. The swing gate is a pivoting flap mounted on a horizontal axis, which swings in downstream direction. Since both gate types are investigated in this work, the movement of the gates is defined in the following to ensure a comparability of the tests between the different gate types.

The lift gate was implemented as a constantly accelerated gate. This implementation was chosen to compare with the Lauber & Hager’s (1998) opening criteria \( (t_{LH}) \) which was formulated for such a gate type, and constantly accelerated lift gates are common in existing test facilities.

\[
t_{LH} \sqrt{\frac{g}{d_0}} = 1.25
\]

The calculation of the position of the gate edge for the constantly accelerating vertical gate is as follows:
\[ \text{LiftG}_{\text{open}} = \frac{1}{2} at^2 \]  

(4)

where \( a \) is the linear acceleration of the gate. At \( t = t_{\text{open}} \), \( \text{LiftG}_{\text{open}} \) is considered completely open when \( \text{LiftG}_{\text{open}} = d_0 \). Consequently, equation 4 can be rewritten and rearranged as follows to yield the acceleration, which is necessary to open the gate completely in the predefined opening time \( t_{\text{open}} \):

\[ a = \frac{2d_0}{t_{\text{open}}^2} \]  

(5)

The swing gate was implemented with constant angular velocity. This implementation was chosen since the physical tests show, that a constant angular speed represented the motion of the gate. To ensure the comparability of the lift and swing gate tests, the vertical distance between gate edge and bottom of the flume (\( \text{SwingG}_{\text{open}} \)) was maintained the same between the gate types at each time step.

\[ \text{SwingG}_{\text{open}} = l_{\text{gate}} - l_{\text{gate}} \cdot \cos(\varphi) \]  

(6)

where \( l_{\text{gate}} \) is the distance between the gate edge and pivot axis, and \( \varphi \) is the opening angle of the gate. At the time \( t = t_{\text{open}} \), the gate has a vertical distance \( \text{SwingG}_{\text{open}} = d_0 \) and \( \varphi = \varphi_{\text{open}} \). Equation 6 can be rearranged to calculate the angular velocity (\( \omega \)):

\[ \omega = \frac{\varphi_{\text{open}}}{t_{\text{open}}} = \frac{\cos^{-1} \left( \frac{l_{\text{gate}} - d_0}{l_{\text{gate}}} \right)}{t_{\text{open}}} \]  

(7)

Equation 7 shows that the swing gate opening time is dependent on the gate geometry of the swing gate (Figure 3), which is not the case for lift gates. Data provided in this work was based on a fixed dimensionless gate length: The gate length (distance between pivot axis and gate edge) was always twice the initial impoundment depth (\( d_0 \)).

Figure 3. Dependency of vertical and horizontal gate movement on the gate length

The opening time was determined on the basis of equation 3, defined by Lauber & Hager (1998), since this equation is commonly used to determine opening times for lift gates. All other tests were generated by multiplying the opening time with a dimensionless factor (\( \Phi \)). The opening time of each test version can be calculated as follows:
The factor $\Phi$ was adjusted, as shown in Table 1. The opening time $t_{\text{open}}$ was equal for both gate types, making the experiments comparable, exemplarily lift gate acceleration and swing gate velocity for $d_0 = 0.5 \text{ m}$ are calculated:

### Table 1. Factor $\Phi$, $a_{\text{Lift}}$ and $\omega_{\text{Swing}}$ to reach the pre-described opening time $t_{\text{open}}$, exemplarily calculated for $d_0 = 0.5 \text{ m}$.

<table>
<thead>
<tr>
<th>$\Phi[-]$</th>
<th>0.50</th>
<th>0.75</th>
<th>1.00</th>
<th>1.25</th>
<th>1.50</th>
<th>1.80</th>
<th>2.50</th>
<th>4.00</th>
<th>6.00</th>
<th>8.00</th>
<th>10.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_{\text{open}}(d_0 = 0.5 \text{ m})[\text{s}]$</td>
<td>0.141</td>
<td>0.211</td>
<td>0.282</td>
<td>0.353</td>
<td>0.423</td>
<td>0.508</td>
<td>0.706</td>
<td>1.129</td>
<td>1.693</td>
<td>2.258</td>
<td>2.822</td>
</tr>
<tr>
<td>$a_{\text{Lift}}(d_0 = 0.5 \text{ m})[\text{m/s}^2]$</td>
<td>50.23</td>
<td>22.32</td>
<td>12.55</td>
<td>8.036</td>
<td>5.581</td>
<td>3.876</td>
<td>2.009</td>
<td>0.785</td>
<td>0.349</td>
<td>0.196</td>
<td>0.126</td>
</tr>
<tr>
<td>$\omega_{\text{Swing}}(d_0 = 0.5 \text{ m})[\circ/\text{s}]$</td>
<td>425.2</td>
<td>283.5</td>
<td>212.6</td>
<td>170.1</td>
<td>141.7</td>
<td>118.1</td>
<td>85.05</td>
<td>53.15</td>
<td>35.44</td>
<td>26.58</td>
<td>21.26</td>
</tr>
</tbody>
</table>

Since the opening time and consequently also the acceleration of the lift gate and the angular velocity of the swing gate for each test depends on the factor $\Phi$, but also on the initial water depth and the swing gate length, only the factor $\Phi$ is given in the following figures to characterize the opening time of the gate. $\Phi = 0.50$ describes a gate which opens twice as fast, and $\Phi = 10.0$ a gate which moves ten times slower than defined by Lauber (1997).

### 3. Results

#### 3.1. Vertical Lift Gate

The parameter $RMS E_t/d_0$ (Equation 2) is presented in Figure 4 for $d_0 = 0.30 \text{ m}$, $0.50 \text{ m}$, and $0.70 \text{ m}$. The $RMS E_t/d_0$ error was calculated by comparing the surface elevation lines at each time step with those of the reference wave. The dimensionless time at which the reservoir starts to empty is marked with a red plus in Figure 4. This time was calculated by determining the time where the wave gauge located at the end of the reservoir drops below 95%. The dimensionless time $T_{\text{overflow}}$ is indicated by a blue plus. The time $T_{\text{overflow}}$ is the time elapsed at which the wave gauge located directly in front of the overflow basin detects a water level of more than 5% of the initial impoundment depth. Generally, the $RMS E_t/d_0$ was shown to be:

1. Zero in the first time step since the initial setup is the same for all tests.
2. Considerably higher in the first time steps than in the later ones, this phenomenon can be justified by the presence of the gate. The gate prevented the water column from collapsing in the first time steps and consequently the error was high in the first time steps.
3. Higher for slower gate opening times. While the instantaneous (reference) dam-break wave immediately propagated through the flume, the slower gate opening resulted in greater interaction between the gate and the wave.
4. Decreasing for later time steps. As soon as the gate was fully open, it no longer influenced the wave. The error remained at a relatively constant level before further decreasing. As already discussed, the high errors in the first time steps are caused by the gate, until the gate is opened the wave can propagate similar to the reference wave but was delayed due to the gate opening process. At the end of the simulation, the error began to further decrease. This was caused by the discharging flume (blue plus marker) and the decreasing water level in the flume. Since the error was calculated by the distance between the two water elevation lines this distance decreases when the water level decreases but is divided by the initial impoundment depth $d_0$, so the dimensionless error decreases as soon as the positive wave reaches the end of the flume.
5. Overall increasing with increasing initial impoundment depth $d_0$.  

$$t_{\text{open}} = t_{HL} \times \Phi$$  

(8)
The parameter $RMSE_x/d_0$ is presented in Figure 5 for $d_0 = 0.30$ m, $0.50$ m, and $0.70$ m. The $RMSE_x/d_0$ error was calculated by using the wave gauge time series of each wave gauge and compared with the time series of the wave gauges measuring the reference wave (see Equation 1). Note that $RMSE_x/d_0$ is a mean error from the first time step to $T_{overflow}$. It was observed that $RMSE_x/d_0$:

1. Increased as the gate opening time increased. This effect can be explained by the fact that the gate hinders the wave in the first time steps and the wave was always offset from the reference wave.

2. Was greatest close to the gate and decreases with increasing distance. The reference wave started to propagate in the first time step, while the present wave is kept in place by the gate, so errors have to be high in the near field of the gate.

3. Overall increases for increasing initial impoundment depth $d_0$.

The parameter $RMSE_t/d_0$ is presented in Figure 6 for $d_0 = 0.30$ m, $0.50$ m, and $0.70$ m. Following descriptions are applicable for all three $d_0$-values. It can be seen that the $RMSE_t/d_0$:

1. Is significantly higher in the first time steps, as in the later ones but only for those four tests which are the ones with gate opening times of at least four times slower than the opening time defined by Lauber (1997), so $\Phi \geq 4.0$ (see Equation 3).
2. Is on a rather constant very low level for all gate openings faster than $\Phi < 4.0$. Consequently, the gate seems to not, or only minimally influence the evolving bore.

3. When the discharge starts, the error increases. This phenomenon is already explained in section 3.1. and is due to the all over all decreasing water level in the flume.

![Figure 6. RMSE/\(d_0\) values of swing gates opened in different times for initial impoundment depth \(d_0 = 0.3\,m\) (left), \(d_0 = 0.5\,m\) (middle) and \(d_0 = 0.7\,m\) (right).](image)

The parameter $\text{RMSE}_{x/d_0}$ is presented in figure 7 for $d_0 = 0.3\,m$, as well as $d_0 = 0.5\,m$ and $d_0 = 0.7\,m$. Following descriptions are applicable for all three $d_0$-values. It can be seen, that $\text{RMSE}_{x/d_0}$:

1. Is higher the slower the gate moves, but only for tests with very slow gate movement ($\Phi \geq 4.0$), since the tests run with faster gate movements are almost not effected by the gate. In general, the higher errors for slower gates can be explained by the gate itself which hinders the wave in the first time steps and leads to an offset between the present wave and the reference one.

2. Is in the near field of the gate significantly higher for the slow-moving gate tests ($\Phi \geq 4.0$) and decreases with increasing distance. The reference wave starts to propagate in the first time step, while the present wave is kept in place by the gate, so errors have to be high in the near field of the gate.

3. Is decreasing in downstream direction as bigger the distance to the gate is. This effect has already explained in section 3.1.

4. Overall increases with increasing initial impoundment depth $d_0$.

![Figure 7. RMSE_{x/d_0} values of swing gates opened in different times for initial impoundment depth \(d_0 = 0.5\,m\) (left), \(d_0 = 0.3\,m\) (middle) and \(d_0 = 0.7\,m\) (right).](image)
4. Discussion

4.1. Comparison of Gate Types

This section serves to point out the differences between the two gate types. The horizontal movement of the swing gate appeared to reduce the error associated with the dam break wave as relatively low errors were observed with reasonable gate opening speeds. Lift gates always had an influence on the flow, leading to high errors in the near field. Based on the work presented here, it is recommended—in case of planning and building of new dam-break test facilities—to select a swing gate where possible. Apart from the positive characteristics of a swing gate, the dependency of the error on \( X \) and \( T_{\text{open}} \) values are comparable to those of lift gates, but errors are significantly smaller when comparing the same opening times for lift and swing gates. At this point it has to be mentioned, that the numerical model cannot take adhesive forces between upstream swing gate surface and water into account. This shortcoming of the numerical model is discussed in section 4.3. In reality, the positive effect of the horizontal swing gate movement might be reduced by these forces, but only for very fast-moving swing gates.

4.2. Practical Application of the Revisited Opening Criterion

The opening criterion by Lauber & Hager (1998) for lift gates prescribed maximum opening times for a fixed \( X \)-position downstream. However, their criterion is not a priori valid for gates with different motion characteristics. Additionally, Lauber & Hager (1998) opening criterion leads to error levels which are about 1% at \( X = 10 \) (where Lauber defined his criterion, see Figure 8 and \( \Phi = 1.0 \)). The error level prescribed by Lauber & Hager (1998) at \( X = 10 \) may be too conservative in many laboratory settings as high acceleration might be too costly to achieve. If a higher error level is tolerable and the distance between gate and test section can be increased (requires a sufficiently long flume), significantly smaller gate accelerations, or angular velocities respectively, can still be viable. For example, if the test section can be placed at \( X = 15 \) and an error of 2.7% (so 1.35 m/s at \( d_0 = 0.5 \, m \)) is tolerable, \( \Phi \) increases from 1.0 to 6.0. Following Table 1, this leads to a significantly lower necessary gate acceleration (from 12.55 m/s\(^2\) to 0.35 m/s\(^2\)). Since acceleration is proportional to force, a drive of the slower moving to be designed gate can be approximately 35 times less powerful than estimated with the Lauber & Hager (1998) criterion, but the flume needs to be 2.5 m longer.

\[ \text{RMSE}_{x/d_0} \]

Figure 8. \( \text{RMSE}_{x/d_0} \) values of lift gates opened in different times for initial impoundment depth \( d_0 = 0.5 \, m \).
4.3. Limitations of the Numerical Model

The numerical model applies a mesh-free, particle-based algorithm to solve the Navier-Stokes-Equations. As a first limitation, surface friction was not considered by the numerical model. It is debatable whether the omission of friction in the numerical model has a significant influence on the test results. Firstly, friction has to be separated into friction between water and bottom boundary and into friction between water and gate surface. Bottom friction leads to a deceleration of the wave, and consequently, the wave propagates slower the greater the friction. This implies that friction has a positive influence on the required gate velocity. The higher the bottom friction, the slower the gate can be moved without negatively affecting the wave. Since no friction is considered in the numerical model, the wave was propagating faster than in reality. The error diagrams (Figure 4-7 and 8) for lift and swing gates are based on such a fast-moving wave, which means that the error in reality would likely be smaller than predicted by the numerical model. Consequently, the frictionless flume assumed in this work does not diminish the errors estimated here.

The frictionless gate was a simplification whose influence on the wave cannot be quantified without further numerical tests. The DualSPHysics code allows to prescribe boundary viscosity to a fixed boundary. This boundary viscosity acts like a predefined friction between gate surface and water. However, it is problematic to set this boundary viscosity correctly, since there is no possibility to change the boundary viscosity value into a common roughness value. Additionally, further numerical tests would be necessary to produce calibration and validation data. Notwithstanding, the influence of the gate roughness on the wave would be expected to be negligible. The gate moves rapidly but only for a short period of time, and water has a high inertia due to its high density. It is likely, that the impact time of the gate is too short to result in significant modulations on the wave due to friction.

It can be assumed that there are adhesive forces between the fluid and the swing gate's upstream surface. DualSPHysics cannot consider these forces in the actual version. Future development of the software might need to look into these aspects prior to answering this kind of questions.

It can be seen in the error plots (Figure 4-7), that the higher the initial impoundment water level, the higher the error values. It is unclear why this discrepancy occurred. Within this study only three different impoundment depths are investigated, further investigations are necessary to understand the influence of the impoundment depth and to scale the error.

The influence of the swing gate geometry on the wave is currently unknown, since it has not been investigated within this study. All investigated swing gates had the same size in relation to the initial water depth, that is, the pivot point was always located \( 2d_0 \) above the bottom of the flume (see Figure 3).

5. Conclusion

The extensive set of numerical simulations showed that the following variables need to be considered in the error estimation of a dam break wave, potentially hindered by a slower-than-ideally moving gate: The opening time of the gate \( T_{open} \), the initial impoundment depth \( d_0 \), the distance between gate and test setup \( X \), and the allowable error \( RMSE/d_0 \) at position \( X \). The following general statements can be made about temporal and spatial error development:

1. The choice of a tolerable error level and an ideally distant downstream position allows to reduce constructional effort imposed on gate drives, or in turn, for high gate accelerations. This leads to a smaller drive to move the gate and consequently saves money when planning and building a new dam-break test facility.

2. Existing dam-break facilities do not have impoundment depths significantly greater than \( d_0 = 0.7 \, m \). A potential reason for this is that the weight of the gate increases with increasing impoundment depths and the forces to move the gate becomes enormous when using the Lauber & Hager (1998) criterion. This work proposes, that significantly smaller accelerations can be used to achieve acceptable error levels, dam-break facility with impoundment depths of several meters become realizable.

3. In case of planning a new dam-break test facility, it is recommended to choose a swing gate rather than a lift gate, since low error levels are easier to reach. However, swing gates have the disadvantage that tests with sediment in the flume could potentially hinder the gate motion. Consequently, those gates cannot be used in sediment test applications without further considerations.
5.1. **Outlook**

Beneath the spatial and temporal error diagrams, which are useful tools for dimensioning lift gate facilities and test setups, and the general understanding for error development and its dependencies on different values like $X$, $T_{open}$, $d_0$, and gate type, some more tests and simulations are necessary to answer the following questions. Additionally, the numerical model has to improve at a few points. Critical questions towards furthering the numerical model are:

1. How to implement adhesive forces between swing gate and water to simulated fast swing gate movements?
2. How to visualize error levels to create powerful design diagrams?
3. Why does higher impoundment depth $d_0$ lead to higher relative error levels $RMSE/d_0$?
4. What is the influence of the gate length, respectively the position of the pivot axis in case of swing gates. In the scope of this thesis the pivot axis is always located $2d_0$ above the bottom of the flume.
5. How does an inclination of the flume affect the error levels?

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7. **References**


